WATER RECLAMATION USING AN ENHANCED SCALABLE WASTEWATER TREATMENT PACKAGE UNIT

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BOD_5	-	Biochemical Oxygen Demand (five day test)
CFU	-	Colony Forming Units
DNA	-	Deoxyribonucleic Acid
DO	-	Dissolved Oxygen
DOH	-	Department of Health
NSF	-	National Sanitation Foundation
NTU	-	Nephelometric Turbidity Unit
OWRS	-	Onsite Water Reclamation System
SIWTP	-	Sand Island Wastewater Treatment Plant
TSS	-	Total Suspended Solids
UV	-	Ultraviolet
VSS	-	Volatile Suspended Solids
WTPU	-	Wastewater Treatment Package Unit

Chapter 1
Introduction

Simply put, water is a precious quantity. We depend on water to grow crops, raise livestock, produce electricity, manufacture an enormous variety of products, and most importantly sustain life. Unfortunately this precious quantity is becoming increasingly scarce due to the demands of society.

While people living in rural areas do not have much difficulty in finding the water they need, large city populations find this challenge much more daunting. Most large cities have already exploited all the nearby water resources and are forced to expand their search for water, often at a great expense. According to a report from the United Nations in 1989, half of the total population in the world will be living in cities by the year 2000. Additionally, the number of cities exceeding 1 million residents is expected to be over 400 by the end of this century. (EPA, 1992). This is a trend that is especially true in the United States. Already in 1966, the U.S. Census Bureau estimated that two thirds of the U.S. population lived in urban areas and of the remaining one third, only 8% truly lived in a rural setting (Wright, 1966). Thus, the search for suitable water effects the majority of us.

In order to sustain water supplies to meet our growing worldwide demand, two primary methods are suggested. First, we must protect the current water sources from contamination. Secondly, we must learn to fully utilize the water that we currently take from these water sources. It is our commitment to these two methods that will determine the continued availability of the water supply that we all so heavily depend upon.

One of the ways in which our current water sources are contaminated is through the use of cesspools and septic tanks. Cesspool systems leach water that has not been treated into ground water aquifers that communities may depend on. Additionally, these systems leach higher levels of soluble phosphorous and nitrogen than is recommended for drinking water supplies. This problem could lead to making a suitable aquifer for a drinking water supply no longer available for such a purpose. A November 1997 survey in Hawai'i determined that over 218,000 cesspools still exist in the major counties, with approximately 158,000 located in the City and County of Honolulu (Island of Oahu). Even if Hawai'i switched to septic tanks, which are a definite improvement over cesspools, contamination to the groundwater can still occur. Septic tanks have been shown to be the leading cause of groundwater contamination (nitrate and phosphorous contamination) in some cities, and this problem is widespread in the United States. It is estimated that approximately one third of the nation's sewage is disposed of through septic tanks. (Harman et al., 1996)

A possible method to avoid or prevent groundwater contamination is to replace cesspools and septic tanks with small-quantity wastewater treatment package units (WTPU's). This is a small tank (approximately the same size as a septic tank) that could replace a septic tank or a cesspool at a residential unit. This unit reduces the total amount of suspended solids in the wastewater, reduces the biochemical oxygen demand, and could reduce phosphorous and nitrogen levels. Then, if the water were recharged into the ground, few if any effects to the surrounding groundwater would arise. A related thesis (McNair, 1999) further explores the performance of a single-family wastewater treatment package unit.

The other method to sustain current water supplies is to fully utilize the water that is currently taken from these water sources. In order to do so, treating wastewater to make it available for reuse is mandatory. This is especially true in urban areas where reclaimed water use could dramatically reduce fresh water demand. Figure 1shows the primary demands for fresh water in the United States (EPA, 1992). This graph shows that only 12% of the total fresh water demand in the U.S. is due to public and domestic use, however, this usage is much more concentrated geographically than some of the other major water uses. Concentrated water use in a given location is the major challenge for a society as population growth in large cities continues to increase.

Figure: 1 US Fresh Water Demands by Major Uses, 1985

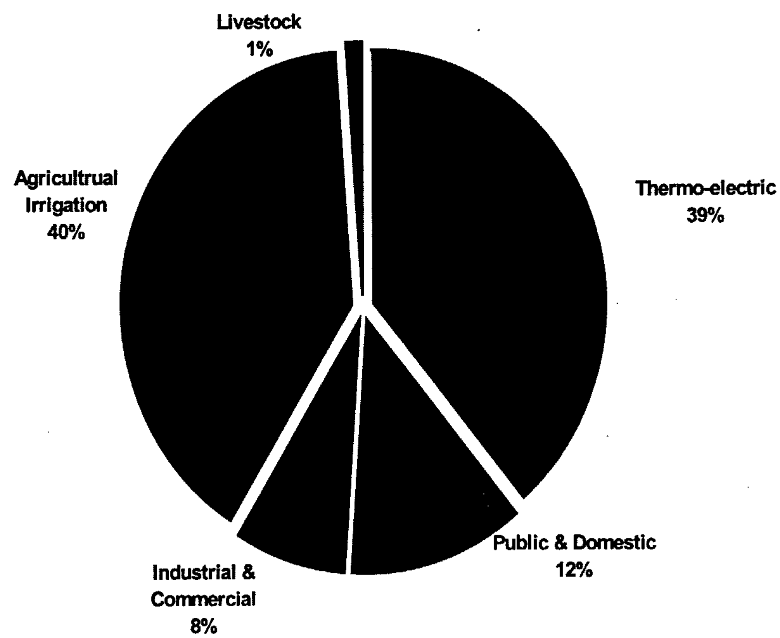
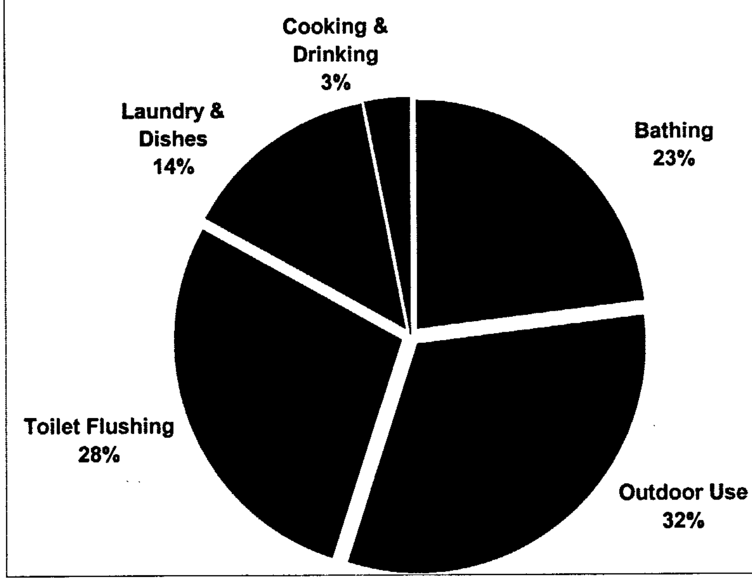


Figure 2 is another compelling graph showing the reuse possibilities in a residential setting. This graph shows the breakdown of residential water use by type of use. Although these statistics may vary due to location and season, they show the general pattern of water usage. For example, outdoor use and toilet flushing comprise approximately 60% of water utilization at a residential unit. Both are types of use that can

use reclaimed water. It is staggering to think that as much as 60% of all water used in a residential setting could be reclaimed water. Furthermore, the burden on the potable water supply for a city would be dramatically reduced if just outdoor water uses and toilet flushing used reclaimed water.

Therefore, if a residential wastewater treatment package unit can adequately treat residential wastewater, then the possibility for onsite residential reuse exists. This Thesis focuses on residential reuse using a small quantity wastewater treatment package unit. Using the effluent from a WTPU (provided by BEST Industries), we attempted to create reclaimed water acceptable to meet the highest reuse standards in the State of Hawai'i. The possibility of creating reusable water at a residence is desirable for many reasons. By reducing potable water usage at a residence, onsite water reclamation could cut down on drinking water cost for a residential home, and would be less taxing on already overburdened drinking water supply systems. Additionally, it could reduce the infrastructure costs and management problems associated with piping sewage to a large-scale, publicly owned wastewater treatment plant. Unfortunately, onsite water

Figure: 2 Average Residential Water Usage by Type of Use



reclamation also has several potential drawbacks such as excessive cost, state and federal regulations, and the demanding maintenance and care of the system.

As this thesis will explain, the cost necessary to meet monitoring requirements to create reclaimed water is currently the primary hindrance to making onsite water reclamation possible. Therefore, changing the current monitoring regulations is the only option to making onsite water reclamation possible. This is the case because even if the water in the WTPU and the following filtration/disinfection steps could always produce high quality water, the current State of Hawaiʻi regulations still insist that continuous monitoring equipment be in place. Therefore, since continuous monitoring equipment is not cost effective, this type of system is completely infeasible. But the fact that regulations must be changed before onsite water reclamation can be possible is not much of an impetus to spur lawmakers to change laws. Obviously, there must be many assurances that public health standards currently in place would not be lowered if the laws were revised to accommodate this process. Therefore, a paradox exists. Current monitoring regulations are the primary stumbling block to making this system feasible, and the responsibility to makes such a system possible and feasible rests on the shoulders of the inventors and manufacturers; although inventors and manufacturers do not have the ability to change regulations. This is the purpose of this study, to wrestle with this paradox. Ultimately, it is my goal to provide insight and new suggestions to the dilemma of making an onsite reclamation system more reliable, and consequently more desirable in the eyes of regulators and legislators.

Chapter 2
Background

2.1 Current Regulations

Demonstrating the technical feasibility of reclaiming wastewater at individual homes and using it for reuse purposes is the desired result from this study. Before further discussion on possibilities for achieving this goal, however, we must first discuss some of the limitations for water reuse in a residential zone. Some of these limitations result from the Hawai‘i State regulations for reclaimed water.

The Hawai‘i State Department of Health (DOH) has prepared a manual titled, "Guidelines for the Treatment and Use of Reclaimed Water" (1993). In this manual, the DOH outlines the uses for reclaimed water, some of which apply to a residential area. The following is a shortened list of reclaimed water uses allowable by the DOH that may be applicable to a residential setting:

1. Residential property where managed by an irrigation supervisor.
2. Flushing toilets and urinals in types of buildings and institutions approved by DOH and where counties have adopted a provision in their plumbing code pertaining to the use of a dual water supply within a building.
3. Supply source for decorative fountains if the recirculating water does not support growth of microorganisms from the surrounding environment that could infect either the respiratory or digestive system of mammals.
4. Washing of hard surfaces (e.g. parking lots and sidewalks).

   (Hawai'i State DOH, 1993)

Already based on the DOH guidelines for water reuse, reclaimed water around a residential unit will be difficult. Point #1 from above, stated that residential use of reclaimed water requires an irrigation supervisor. The probable intent of this statement is for large estates or ranches to have an on-site employee monitor the use and quality of the reclaimed water being used. Since this is impractical for a single household (which is the focus of this study) exceptions or revisions to this first rule need to be implemented before this system could be operated in a single residence setting. Points 2, 3 and 4 from above, although they are not as directly related to residential water reuse, are still applicable in a residential setting. The problem with these uses, however, is the additional cost associated with using the reclaimed water (i.e. replumbing the entire house) and the impracticality of using reclaimed water for such a purpose (i.e. installing a fountain is one's backyard). Of the last three opportunities for reuse, perhaps the most readily applicable use is point #4, to use reclaimed water for cleaning and washing of hard surfaces, such as sidewalks and driveways.

There are other problems encountered in the DOH guidelines using an onsite water reclamation system (OWRS) to create reclaimed water for a single household. The above list is only the state's policy on where reclaimed water could possibly be used. Also included in the DOH guidelines are requirements for the use of reclaimed water. The following list is a compilation of the requirements for using reclaimed water that currently apply to using water in a residential setting:

1. Signs shall be posted where reclaimed water is used.
2. Adequate measures shall be taken to prevent ponding of reclaimed water.

3. Reclaimed water shall be managed to avoid conditions conducive to proliferation of mosquitoes and other disease vectors.
4. No discharge, runoff, or overspray shall extend beyond the approved use of area boundaries.
5. There can be no irrigation within a minimum of 50 feet of any drinking water supply well.
6. The outer edge of the impoundment shall be located at least 100 feet from any drinking water supply well.
7. Drainage shall be controlled to prevent reclaimed water from coming within 50 feet of a drinking water supply well.

(Hawai'i State DOH, 1993)

This list of current requirements for reclaimed water use brings up a myriad of questions when trying to apply these requirements to an OWRS. First, if the requirement to place signage were strictly enforced, convincing a homeowner to place such a sign in their front lawn would be difficult. This requirement may be less of a problem for residences in a more rural setting; however, a sign of this nature may prove to be an uninviting decoration in any condition even at rural homes. Other problems with this list of regulations concern the water boundaries for reclaimed water use. In a residential area, monitoring the use of reclaimed water only within the allowed boundaries would be difficult. Although boundaries are not too difficult to enforce at some reuse sites, they would be difficult to strictly enforce at a residential unit. For example, a golf course, where the irrigation area is large and there is usually some unused space between adjoining properties, makes the risk of overspray potentially less hazardous. In contrast,

a residential unit usually has a much smaller irrigation area making the possibility of overspray or leakage into a heavily trafficked public land space (such as a sidewalk, street or neighboring yard) much more likely.

Obviously, the current guideline is not written with the expectation that single residential units will be making their own reclaimed water. Therefore, it can be expected that the requirements in the DOH guidelines will have to be revised before an OWRS could become a reality. Also, it should be kept in mind that the guidelines are only guidelines. Thus, as long as the public safety is maintained, the guidelines may be followed less strictly than how they are written. Later in this report, a likely policy to monitor an OWRS will be discussed. Some of the guidelines currently in place may not apply to an OWRS while others may have to be revised to accommodate the new system.

2.1.1 R-1 Quality Water

The reclaimed water standards for Hawaiʻi are also outlined in the DOH manual "Guideline for the Treatment and Use of Reclaimed Water". The state has defined three levels of reclaimed water quality: R-1, R-2 and R-3. Simply put, R-3 quality water has been through a secondary treatment process, R-2 water has undergone secondary treatment and disinfection, and R-1 water has been treated through a secondary treatment process, filtration, and disinfection. For the purposes of water reuse in a residential setting, R-1 quality water is always required with one exception. (This exception is that R-2 quality water can be used in a residential setting if subsurface irrigation is used) Therefore, all references to reclaimed water in this report will be specific to R-1 quality

water unless otherwise stated. According to the State DOH, R-1 quality water is defined as follows:

> "R-1 water (significant reduction in viral and bacterial pathogens)" means reclaimed water that has been oxidized, filtered, and disinfected to meet the following criteria:
>
> A. A disinfection process that, when combined with the filtration process, has been demonstrated to reduce the concentration of plaque-forming units of F-specific bacteriophage MS, or polio virus, per unit volume of water in the wastewater that will occur during the reclamation process. A virus that is at least as resistant to disinfection as poliovirus may be used for purposes of the demonstration.
>
> B. Fecal coliform bacteria densities as follows:
>
> (1) The median density measured in the disinfected effluent does not exceed 2.2 per 100 milliliters utilizing the bacteriological results of the last seven days for which analyses have been completed; and
>
> (2) The density does not exceed 23 per 100 milliliters more than one sample in any 30-day period; and
>
> (3) No sample shall exceed 200 per 100 milliliters.

(Hawaiʻi State DOH, 1993: Addendum No. 1, October 27, 1998)

This list is the basic requirement for R-1 quality water, although other requirements also exist due to regulations placed on the processes necessary to create R-1 quality water.

First, there are requirements for filtration of secondary effluent that must be met. In order to use secondary effluent for reuse purposes, the secondary effluent must have a turbidity of 5NTU or less. Additionally, after filtration, the water must have a turbidity of 2NTU or less. If either of these two turbidity levels is not met, all non-compliant water must be diverted to another purpose other than reclaimed use. These strict requirements make the goal of creating R-1 quality water at a residential unit very difficult. Already regulations require two continuously recording turbidimeters to monitor the effluent water from the WTPU and the filtration process. This process greatly increases the cost of a home reclamation system, thus, making this process less desirable for the average homeowner.

Many requirements also exist for the disinfection step of creating R-1 quality water, specifically when using ultraviolet light to accomplish the disinfection process. First, there are additional water quality standards that must be measured. The minimum allowable wastewater transmittance is 55%. This will likely be met if the wastewater turbidity is less than 5NTU; but since it is a required water quality parameter, it must be measured. Thus, a meter to measure transmittance is required in the overall system. Secondly, UV disinfection requires a great deal of maintenance compared to other disinfection processes. As a minimum, the DOH guidelines require alarms to monitor:

(1) Flow rate

(2) Fluid transmittance

(3) Turbidity

(4) Liquid level in UV disinfection channels

(5) Status of each UV bank, on/off

(6) Status of each UV lamp, on/off

(7) UV intensity measured by at least one probe per bank.

(8) Lamp age in hours

A quick observation of the above list lends great insight to the amount of monitoring required before an ultraviolet disinfection process can be operational. This list requires a flow meter, a turbidimeter, a photometer (to measure transmittance) and various electronic float switches, valves, and other electronic devices to monitor the disinfection process. Also, current guidelines require that a water reclamation system have a backup power supply for all electrical systems in the processing of the reclaimed water, especially the UV system. Finally, other requirements for an ultraviolet disinfection system are that the system must be able to deliver a minimum dose of 140 $mW\text{-}s/cm^2$ at an average design flow and 100 $mW\text{-}s/cm^2$ at a peak flow; and that this delivery must be achieved with a minimum of three UV banks in series.

2.1.2 R-2 Quality Water

According to the State DOH, R-2 quality water is defined as follows:

"R-2 Water (Disinfected Secondary-23 Reclaimed water)" that has been oxidized, and disinfected to meet the following criteria:

A. Fecal coliform bacteria densities as follows:

1. The median density measured in the disinfected effluent does not exceed 23 per 100 milliliters utilizing the bacteriological results of the last seven days for which analyses has been completed; and

2. The density does not exceed 200 per 100 milliliters in more than one sample in any 30-day period.

As shown above, the requirements for R-2 quality water are much less stringent than R-1 quality water; however, the uses are much more limited. This is especially true of using R-2 water in a residential area. Currently, the Hawaiʻi State DOH only allows for R-2 water to be used in subsurface irrigation (drip irrigation) and where managed by an irrigation supervisor. Therefore, since R-2 water can only be used for one purpose in a residential setting, the desire to create R-2 water is severely diminished. This desire is further diminished considering the impracticality of drip irrigation using R-2 water. Studies show that the major clogging problems associated with drip irrigation are due to the presence of suspended particles in the irrigation water (Adin, 1987; Asano, 1999). Therefore, since R-2 water is unfiltered, it would make a poor source for a drip irrigation system.

In addition to the microbial requirements for R-2 water, the disinfection process also has some guidelines to follow. (Specific to the testing performed in this study, was chlorination using Trichloro-s-triazetrione tablets.) Therefore, the guidelines for chlorination to meet disinfection requirements for R-2 water are applicable. The State DOH guidelines outline the following three paragraphs for the necessary requirements of a chlorination disinfection process:

Level 2 Chlorination which meets the requirements of disinfection for R-2 water and is exposed to chlorine in a well baffled contact basin or pipeline that provides:

1. A chlorine contact time and residual, either or both of which differ from that cited in paragraph (2) below, that have been shown to the satisfaction of DOH to

reliably reduce the concentration of fecal coliform bacteria so that at some location in the treatment process the median number of fecal coliform bacteria in the effluent, as determined by approved laboratory methods, does not exceed 4 per 100 milliliters, as determined from the bacteriological results of the last seven days for which analyses have been completed, and the number of total coliform bacteria does not exceed 50 per 100 milliliters in any sample; or

2. A theoretical chlorine contact time of 15 minutes or more and an actual modal contact time of 10 minutes or more throughout which the chlorine residual is 0.5 mg/L or greater, and the median number of fecal coliform bacteria in the effluent, as determined by approved laboratory methods, does not exceed 4 per 100 milliliters, as determined from the bacteriological results of the last seven days for which analyses have been completed, and the number of fecal coliform bacteria does not exceed 50 per 100 milliliters in any sample; and
3. Automatic control of chlorine dosage and automatic, continuous measuring and recording of chlorine residual shall be provided. The chlorination facilities shall have adequate capacity to maintain a residual of 2 mg/L.

Note that the chlorination process guidelines in the State DOH manual cite lower concentrations for fecal coliforms than the overall requirements for R-2 water. This discrepancy most likely occurs because the definitions for R-1 and R-2 water were amended in a revision five years after the reuse guidelines were originally written. Therefore, the chlorination process guidelines should reflect the less stringent requirements stated earlier in the definition of R-2 water.

2.2 History and Theory of Filtration

Filtration is probably the oldest form of drinking water treatment, or at least the oldest effective form of treatment. A collection of medical lore written in Sanskrit around 2000 BC directs that foul water should be boiled, exposed to sunlight, filtered and then cooled in an earthen vessel (Baker 1981). Other records from Egyptian, Biblical, Greek and Roman records also describe methods to treat water, some of which discuss filtering water through granular media. Although the mechanics involved in filtration are more well known today, the overall process is still fairly similar. In fact, rapid sand filters are still commonly used as the final clarifying step at most municipal water treatment plants in the U.S. (Vigneswaran et al., 1995). In the United States, the history of filtration begins with Albert Stein, who engineered the first sand filter for municipal use in Richmond VA in 1832 (Baker, 1981). The first attempt to create an upflow filter made of sand and gravel with a reverse flow cleaning system was a failure, but not long after, this sand filtration system, as well as others, were up and running in the U.S.

Filtration is a physical process that separates solids of varying sizes from the liquid. Removing suspended material from the water is the major purpose of filtration (measured as turbidity). Lowering the turbidity of the water is important because the suspended particles can act as a shield and prevent the microorganisms from coming into contact with the disinfectant (usually the following step in water and wastewater treatment). In some cases the particles can combine chemically with the suspended solids, leaving less disinfectant to combat the microorganisms (AWWA, 1984). Thus, removing suspended particles through filtration greatly assists the following process of disinfection. Filtration occurs when water passes through a medium of granulated

material such as sand, anthracite, or granular activated carbon. There are various processes by which the particles in suspension in the water separate out of the water and stay in the filtering medium. Often times the process by which the particle separates from the water is due to particle size, but there are many other physical and chemical forces at work.

The first filtration mechanism is called straining. Straining occurs when particles that are larger than the pore sizes between the filtering medium are trapped in the filter. This filtration mechanism is only effective for the largest particles in suspension since the particles, which are desirable to remove, are much smaller than the filter medium particles in the filter. For example, in this study a sand grains of 0.5mm (500μm) and 0.1mm (100μm) were chosen for the dual-medium sand filter. The pore size for this sand is much greater than the colloidal particles (5μm and less) which this filter is intended to remove to some degree. Therefore, since many of the colloids can be greater than 100 times smaller than the filter medium, there must be additional forces at work in a sand filter to make it effective.

The second type of filtration is sedimentation. Sedimentation occurs within pore spaces of the sand particles where the flow is slow enough to cause the particle to settle out of the flow. This is one reason why the filtration rate in a filter is so important. On one hand, a faster flow through the filter determines the size needed for an overall filter design; however, increased flow rate makes the flow through the filter media pores much more rapid and increases headloss. This in turn limits quiescent flow zones in the sand filter where particles can settle out of the flow. Thus, filtration rate is a vitally important factor in filtration design.

A similar filtration mechanism to sedimentation is impaction. Like sedimentation, some particles settle out of the flow and onto the filter media. Unlike sedimentation, however, impaction occurs when the suspended particle becomes lodged in the filtering media due to the force of the flow causing the particle to impact the filtering media. Consequently, the suspended particle can no longer follow the flow stream.

Another filtration mechanism is interception. Interception occurs when a suspended particle comes in contact with the filtering medium. This contact slows the inertia of the particle, causing it to no longer follow the flow of the water in the filter. Again, filtration rate plays a major role in this mechanism. If the filtration rate is increased, the force needed to overcome the friction force that keeps the particle out of suspension will increase. Once again this particle would be in suspension. Ultimately, the loss of this filtration mechanism due to an increased filtration rate could increase the turbidity of the final effluent.

Adsorption is another major mechanism involved in filtration. Adsorption occurs when particles contact and adsorb (or stick) onto the surface of the filter medium or onto previously deposited material. Adsorption can occur through chemical bonding and interaction, or it can occur through physical forces. Electrostatic, electrokinetic, and Van der Waals forces are some of the physical forces that can attract a suspended particle out of the flow stream. The effectiveness of adsorption depends most on the type of filter medium used and the overall specific surface area of the filter medium.

Finally, flocculation and biological growth are the other processes involved in filtration. Removal of particles through biological growth in the filter is especially important in slow sand filters (Vigneswaran, 1995). Biological growth can be a nuisance

in rapid sand filters, however, causing a major impact on headloss. Biological growth can even cause an increase in turbidity due to the slough-off of biological films in the filter (Asano, 1998). Thus, the desirability of biological growth in a filter highly depends on filter type. Flocculation in a filter is primarily caused by the addition of chemical additives. The addition of chemicals is part of the coagulation/flocculation process that often precedes filtration to increase the particle size of the particles that are desirable to be removed. Proper flocculation time and mixing rate determines the effectiveness in the filtration process. Thus, the coagulation/flocculation process prior to filtration controls the overall effectiveness of flocculation in the filter.

Depending on the purpose of the filtration process (to treat drinking water or wastewater), different filtering mechanisms play the largest role in removing particles from the water. A well-known source, Wastewater Engineering: Treatment, Disposal and Reuse (Metcalf and Eddy, 1991) claims that straining is the most important mechanism for removing particles and consequently all mathematical models are based on this assumption. However, Introduction to Water Treatment (AWWA, 1984) claims quite the opposite. The American Water Works Association claims that straining is the least important filtering mechanism and that adsorption is the predominant factor for removing particles from the water. The primary difference between drinking water and wastewater filtration lies in the analysis of particle size distribution. One study at the County Sanitation Districts of Los Angeles County shows that the turbidity of varying wastewater effluents are largely impacted by the size of particles in the wastewater (Asano, 1998). This study shows that in some cases, the particles in the 5μm range and higher are the primary cause of turbidity. While in other cases, particles in the colloidal

range of 0.01µm to 2µm are mainly responsible for the turbidity. This analysis of particle size is key in determining the primary filtration mechanism to remove suspended particles. If the particles are larger (as is the case in wastewater treatment), the mechanism of straining will play a much more important role in removing suspended particles. In contrast if the particles are in the smaller range (typical of drinking water), then adsorption is probably the predominant mechanism to remove suspended particles.

Mathematical models are rarely used in designing a filter. Most of the time, filter design is based on previous experience. Previous experience states that there are two main types of sand filters: slow sand filters and rapid sand filters. Slow sand filters were first introduced in the U.S. in 1872 (AWWA, 1984). Slow sand filters rely on fine sand and a sticky mat of suspended material called a "Schmutzdecke". This layer forms on the surface of the sand bed and may take as long as two weeks to form before the filter can effectively be used to remove turbidity. Formation of a schmutzdecke requires very low flow rates (0.05 gpm/sf). Consequently, since the schmutzdecke is vitally important to a slow sand filter, the filter is not regularly cleaned. Instead, slow sand filters are cleaned by scraping off the schmutzdecke and the top 6 inches of sand only after a noticeable change in the effluent quality occurs. The sand is then washed and replaced and a new schmutzdecke forms. Slow sand filters are only occasionally used to treat drinking water and are not commonly used in wastewater treatment.

The other major type of filter is rapid sand filters. A rapid sand filter can accommodate a flow rate 40 times that of a slow sand filter (2gpm/sf). Additionally, rapid sand filters can be backwashed since they do not rely upon the formation of a schmutzdecke to remove suspended particles. (A mat layer is formed in rapid sand filters

which slightly improves filter performance, however, it is not vitally important to the overall performance of the filter.) Backwashing is a process where water is forcefully passed through the filter in direction opposite of the normal flow. Over time (usually between 15-36 hours at the operational filtration rate) a sand filter will fill with collected material and dramatic loss in head will be observed. Backwashing removes nearly all the collected particles in a sand filter and cleans the filter media of collected material. This cleaning allows for the filter to once again be useful at reducing the turbidity. The backwash process usually uses between 3-6% of the total amount of water filtered (Vigneswaran, 1995). In addition, the backwash water is returned to an earlier process in the overall treatment of the water.

Within the classification of rapid sand filters, two main subcategories exist: gravity and pressure rapid sand filters. Simply put, the flow through a gravity filters is due to the force of gravity. For a pressure filter, the flow through the filter medium is due to the amount of pressure applied. Interestingly, the filtration rate for both types of filters is about the same. The major advantage to a pressure filter is that air binding (or the creation of a negative head within the filter) will not occur. Since pressure filters are within an enclosed container, however, they cannot be observed for problems until an increase in turbidity is noticed. Therefore, due to their ease of maintenance and minimal operational cost, gravity rapid sand filters are the most common type of filter in use today.

There are many types of filter bed options available depending upon the type of use and the characteristics of the water to be treated. Bed types range from shallow, mono-medium stratified beds, to mono-medium unstratified beds, and even to dual and

multi-medium stratified beds. Bed depths can range from 6 to 84 inches and effective grain size can range from 0.2mm to 4mm (Metcalf and Eddy, 1991). The effectiveness of a filter highly depends on its type and the characteristics of the water to be filtered, thus, pilot studies are always recommended to aid in filter design. In general, however, turbidity removal efficiencies of 70% can be achieved for waters with an initial turbidity of 5.0NTU or greater (Asano, 1998). In addition, the effective turbidity removal generally decreases as the turbidity of the influent water decreases.

2.3 History and Theory of Ultraviolet Disinfection:

The germicidal disinfection capability of ultraviolet light has been recognized since the early 1900's. In fact, the U.S. has recorded its use at municipal drinking water systems since 1929 when an eight-lamp system was installed in Berea Ohio. (Hansen & Sawyer, 1992) Due to the evolution of chlorination technologies however, UV disinfection all but disappeared. Disinfection using chlorine is now the most common form of disinfection in the world. Approximately 60 percent of the 20,000 municipal wastewater treatment plants in North America use Chorination, 24% use Chlorination/dechlorination, 15% use UV irradiation and 1-2% use ozonation. (Reed, 1998).

Disinfection using chlorine is preferred over other methods for many reasons. First, using chlorine has traditionally been the most cost-effective form of disinfection. Since cost drives most issues, this method is obviously the most common. Secondly, chlorination is an established technology hence design issues are well understood. This is an especially important decision factor for small communities needing to build a wastewater treatment plant. Finally, chlorination processes easily meet disinfection standards, while other methods require a greater level of system monitoring and management.

In the last 10+ years however, using ultraviolet light to disinfect drinking water and wastewater has grown rapidly. This resurgence of a relatively old disinfection process is due to a variety of reasons, probably the largest reason for the renewed interest being cost. The development of new lamps and ballasts has reduced the cost of UV disinfection systems. Also, as UV disinfection system become more popular, design costs are

reduced. Perhaps the primary reason for UV disinfection systems being more cost effective may be due to the increase cost for chlorination disinfection. New regulations from state and federal authorities are causing this increase. In 1993, many states adopted the Building Officials and Code Administrators National Fire Code. This code included some stringent measures for handling hazardous materials. Two hazardous materials named in the Code were chlorine gas and sulfur dioxide, chemicals that are used in chlorination/dechlorination processes. When the additional costs associated with the new safe storage and handling procedures are taken into consideration, chlorine disinfection doesn't look as attractive as it used to. Consequently, a growing number of wastewater treatment facilities are opting to use UV technology instead of chlorination for disinfection purposes.

Disinfection using ultraviolet radiation is effective because of its damaging effects on DNA. In particular, UV light between the 230nm to 270nm spectrum (Reed, 1998) is absorbed by DNA cells and inactivates the biological organism rendering it unable to proliferate and thus, unable to cause disease. This process is a physical process. It is different from chemical processes (such as chlorination) where the chemical disinfectant disrupts cell functions, which kills the organism.

UV light is produced when mercury vapor is excited by electricity. This type of lamp, when manufactured at a pressure of approximately 10^{-2} torr (1/760 of standard atmospheric pressure), will produce a monochromatic wavelength with 85% of the lamp's output at 253.7nm (Linden, 1998). This type of lamp is called a low-pressure, low-intensity UV lamp. Low-intensity lamps usually operate at a power of 65 (Reed, 1998) to 85 watts, are usually 2.5 to 5 ft in length, and 0.6 to 0.8 inches in diameter

(Metcalf & Eddy, 1991). Low-intensity lamps are effective at producing wavelengths well suited for germicidal purposes, yet the low intensity of the lamp can make treatment of large quantities of water difficult. In the city of Calgary in Alberta, Canada, the largest low-intensity lamp wastewater treatment facility in the world operated 11,520 lamps treating a flow of 265 million gallons per day. This system would be a maintenance nightmare.

Medium-pressure, high-intensity UV lamps is the most common system now being used for larger flow treatment plants. High intensity mercury vapor lamps operate at a pressure of approximately 10^3 torr and use between 5,000 and 30,000 watts per lamp (Reed, 1998). A high-intensity lamp is much less efficient at producing germicidal wavelengths: 20 to 40% in the germicidal range (Linden, 1998). Due to the much higher operating power, however, the high-intensity lamps can generate 50 to 80 times the germicidal UV output than a low-intensity lamp (Asano et al., 1998). For this reason, one high-intensity lamp can replace many low-intensity lamps. This is extremely advantageous from a maintenance point of view. Therefore, as a typical rule of thumb, medium-pressure, high-intensity lamps are now recommended for wastewater treatment plants with flows greater than 10mgd (Reed, 1998).

An another important fact to know about UV disinfection is the role that the wastewater quality has on the effectiveness of the ultraviolet light to disinfect the water. Since UV radiation is only effective if the light penetrates the cell wall of the targeted organism, anything that will protect or shield the organism from the UV light hinders the effectiveness of this disinfection process. Therefore, the total suspended solids (TSS) concentration plays a major role in the ability of UV light to effectively disinfect the

wastewater. Additionally, even smaller particles than those typically measured in a TSS test can influence the effectiveness of UV disinfection.

TSS is usually measured by collecting all suspended solids on a glass fiber filter with an approximate pore size of 1.6μm. Since bacteria and other colloidal particles can pass through this filter, their mass is not part of the TSS measurement. Even the small colloidal particles approximately the same size as one bacterial organism can temporarily shield the bacteria from the damaging effects of the UV light. The measurement of the absorbance or deflection of the ultraviolet light in the germicidal range is called transmittance. Transmittance is the measurement of the percent of light at the 254nm wavelength that passes straight through a water sample. This is an important measurement because it is an indicator of the required dose necessary for effective disinfection. If the percent of transmittance is high, then this means that there are relatively few organisms, colloidal particles, or organic particles blocking or absorbing the transmission of UV light through the sample. In other words, the intensity throughout the water sample will be relatively high (or close to the actual intensity of the lamp). On the other hand, if the percent transmittance is low, then this means that there are many organisms, colloidal particles, or organic particles blocking or absorbing the transmission of UV light through the sample. Therefore the intensity of UV light in the water will be lower that the actual intensity of the lamp. It will also vary throughout the water sample due to the dispersion of light from bacterial or colloidal adsorption. In conclusion, a lower percent transmittance requires a higher UV dose to obtain inactivation of the water sample.

Different microorganisms are more or less resistant to UV light. In general, the smaller organisms require less of a UV dose to render them biologically inactive. For example, the EPA requirement for viral inactivation of the hepatitis A virus is between 21 and 31 mW-sec/cm^2 (Hansen and Sawyer, 1992). This is a much smaller dose than a dose of 140 mW-sec/cm^2 which is required by the State of Hawaiʻi to obtain levels of fecal coliform that is satisfactory for reclaimed water use (Hawaiʻi DOH, 1993). As we can see from these regulations, some viruses are easier to inactivate than some bacteria. This generalization in size, however, is not always true. For example, the MS2 coliphage, F specific single stranded RNA virus has been shown to be more resistant to UV light than fecal or total coliform bacteria (Braunstein et. al., 1996). Other organisms such as Giardia Lamblia and other protozoan pathogens are considered to be unaffected at most commercially available ultraviolet doses (Hansen and Sawyer, 1992).

When determining the effectiveness of ultraviolet disinfection, it is important to know the received UV dose. A received UV dose is the amount of germicidal UV light that actually reaches a given microorganism. This is different from an applied dose, which is based on the power of the UV lamp. The absorption of UV light by dissolved compounds, suspended colloidal particles and other microorganisms is the cause for this difference.

There are two basic methods for determining a received UV dose. The first common method is a bioassay, which uses directly focused beams of ultraviolet light (produced in a laboratory setting) at a known intensity over a given period of time to measure the amount of deactivation of a selected microorganism in a water sample. The enumeration of the remaining reproducible microorganisms is used to determine the relationship

between the reduction of microorganisms for different UV doses. Then using samples obtained from an operational onsite UV system, the measured reduction can then be compared with the reductions obtained from the laboratory results. Once similar reductions are observed between the two measurements, the relationship between microbial reduction and the UV dose achieved in the laboratory can be used to estimate the received UV dose in the onsite UV system.

The second common method for determining the received dose to a microorganism is the point source summation method. The point source summation method is a mathematical model first developed by Jacob and Dranoff (1970; Blatchley III, 1997) and later applied to UV reactors by Qualls and Johnson (1983). This method determines the energy radiating from any point on the ultraviolet lamp to any point in the lamp reactor. To compute the actual intensity at a given point, the intensity from every point on the lamp is summed to determine the total intensity at a given point. Then an average intensity in the reactor can be determined by averaging the computed intensities at a representative number of points in a cross sectional plane of the UV reactor. The following equation describes the UV intensity at a given distance from a tubular UV lamp (White, 1999).

$$I(r,z)=\sum_{n=1}^{n=N}\frac{S/N}{4\pi(r^2+z^2)}\exp[-a(r^2+z^2)^{1/2}]$$

Where: S = UV energy output and source (Watts)

N = number of point sources into which line source is divided

r,z = coordinates of the point receiver (cm)

a = absorbance coefficient of the wastewater (cm^{-1} to the base e)

Once the average intensity is determined, the average UV dose is calculated by multiplying the intensity by the average time a volume of water remains in the reactor.

Case Study (Braunstein et. al., 1996)

Ultraviolet disinfection is gaining popularity as a disinfection alternative in the production of reclaimed water since UV disinfection has been shown to be an effective treatment method for disinfection even when low microbial concentrations are required, as in strict water reclamation regulations. A study at the University of California, Davis recently tested the effectiveness of using UV disinfection for reuse applications by applying it to filtered activated sludge effluent. The study was performed with two main objectives:

1. Assess the performance of a specific UV system in meeting stringent reuse criteria
2. To compare the results obtained in determining UV dose by the PSS method with results obtained from a bioassay.

The study used a sand filter made by Dynasand. The filter design was a continuous-backwash, deep-bed, granular medium upflow filter using 0.9mm sand with a uniformity coefficient of 1.5. Additionally, the sand filter operated at an applied flow rate of 7.1 gpm per ft^2 and without any chemical addition. Fischer & Porter Ltd. manufactured the UV disinfection system used in this experiment. The designed system consisted of a stainless steel open channel with three UV banks in series. Each UV bank contained four lamps with arc lengths of 148.4cm oriented parallel to the fluid flow direction. Each lamp operated at 26.7 W and was encased in a fused quartz sleeve and completely sealed with

O-rings. The UV disinfection system was operated at a flow of 74gpm with an adjustable outlet weir to keep the height of the water at a constant height.

Testing of this system consisted of measuring total and fecal coliforms, and MS2 coliphage, F-specific single stranded RNA virus. At each test run, and in each UV bank, the experimenters took samples to measure the selected microorganisms. Also, the point source summation method was determined at each test run based on the UV bank configuration and the transmittance.

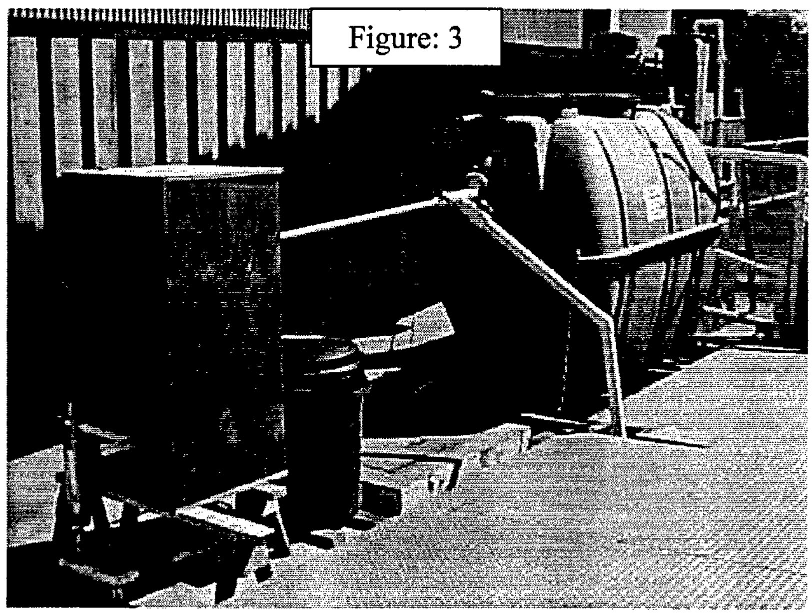
The results of the 22-week study showed that the Dynasand filter met the desired turbidity of less that 2NTU approximately 84% of the testing days and never exceeded 2.5NTU. Also, the filtered effluent yielded an average transmittance of 80.2%. The results from the bioassay showed that reuse standards of less than 2.2 CFU/100mL was met for fecal coliform after a dose of 112mW-sec/cm^2 and 168mW-sec/cm^2 for total coliform. The study also confirmed that MS2 coliphages were more resistant to UV disinfection that the coliform bacteria. The results of the point source summation method showed that an equivalent estimate of the UV dose could be obtained using this method (within a 95% confidence interval).

Another interesting result of this test concerned the maintenance of the UV disinfection system. During the first 50 days of the test the UV lamp quartz casings were cleaned daily. But during the last 90 days of the test, no cleaning was performed on the quartz sleeves at all. The results showed that no difference in coliform counts resulted between the two periods and no buildup was observed on the lamps in either period.

Chapter 3
Methods

3.1 System Construction:

In order to create R-1 quality water, filtration and disinfection processes are required in order to meet the State DOH guidelines. In this experiment to create R-1 quality water a dual-medium, unstratified sand filter was used to fulfill the filtration process. A single-lamp ultraviolet radiation system was used to perform the disinfection process. A picture of the constructed unit, which is attached to the wastewater treatment package unit, is shown in Figure 3.



First, the effluent from the wastewater treatment package unit was diverted from its normal route which discharged all treated effluent back into the main channel of the incoming wastewater for Sand Island Wastewater Treatment Plant. Rerouting was achieved by placing a 2-inch by 1-inch tee onto the effluent discharge pipe. Then a 2-inch valve was installed to control flow returning to the untreated wastewater channel, and a 1-inch valve was installed to control flow to the filtration / disinfection processes (Figure 4).

The 1-inch valve when opened allowed secondary effluent from the WTPU to enter into the sand filtration unit. The sand filtration unit was constructed of a 25-gallon plastic water container with a threaded opening at the base of the container. The plastic container is 14 inches in diameter yielding an inside surface area for the sand filter of 1.1 ft^2. The sand filter was packed with a 4-inch layer of crushed basalt rock (type 3b fine) and overlaid with 5 inches of silica sand (0.5mm) and 7 inches of 90-grit (0.1mm) fine quartz sand. In essence, the sand filter was unstratified since the two sand types mixed thoroughly during backwashes. The only exception was that the upper 1 to 2 inches of the filter seemed to be primarily the 90-grit quartz sand. This sand layer was formed because the force of the backwash fluidized the finer sand more that the silica sand creating a finer top layer. Figure 5 is a picture of the sand filter used in this experiment.

Figure: 4

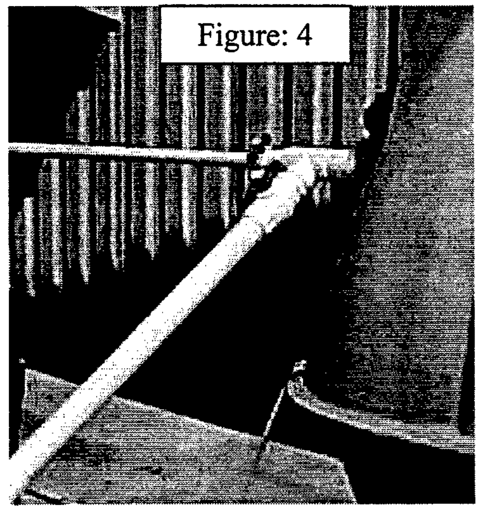


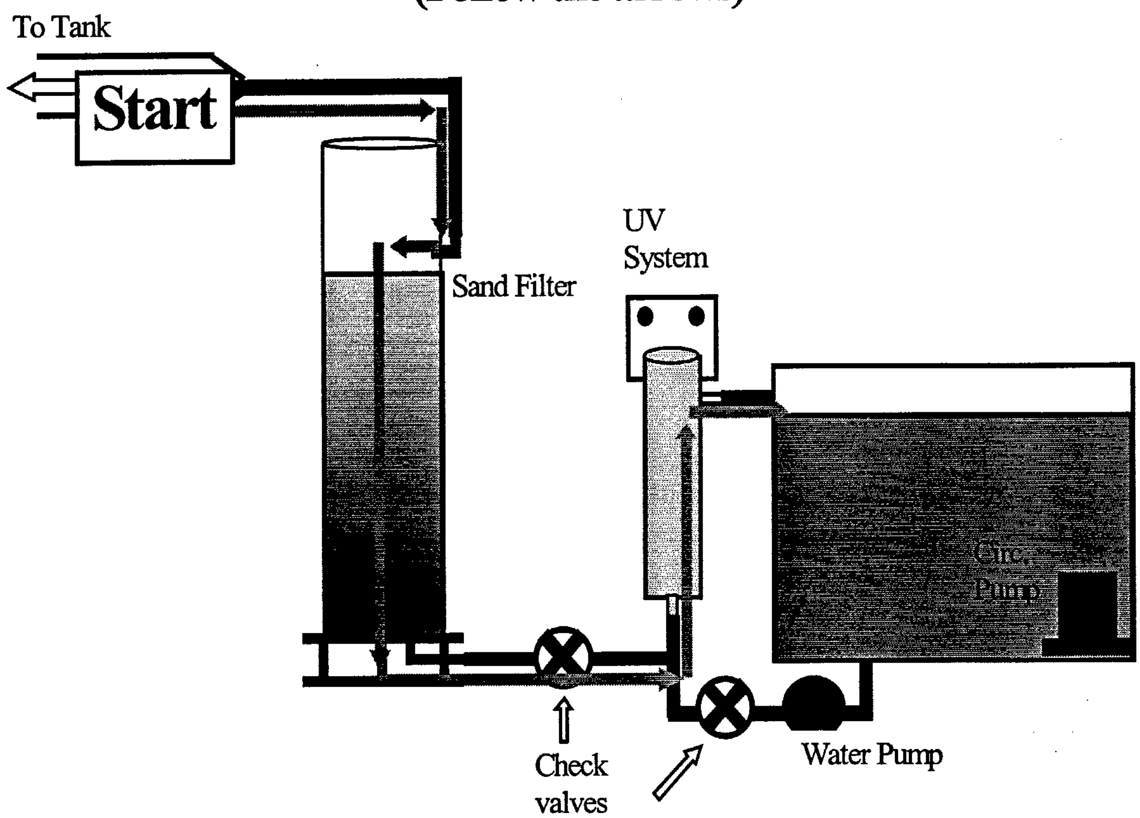
Figure: 5

Water entered the filter from the top of the container through a 1" diameter PVC pipe. To minimize disturbance to the top layer of sand in the filter, the 1" PVC pipe diffused the water entering the filter. The diffusion was achieved by capping the pipe and drilling several holes throughout the last 8 inches of the pipe.

The holes allowed for the water to flow in and evenly spread over the top layer of sand in the filter. After passing through the sand filter, the water flowed via gravity through the

Figure: 6, Initial Loading

(Follow the arrows)



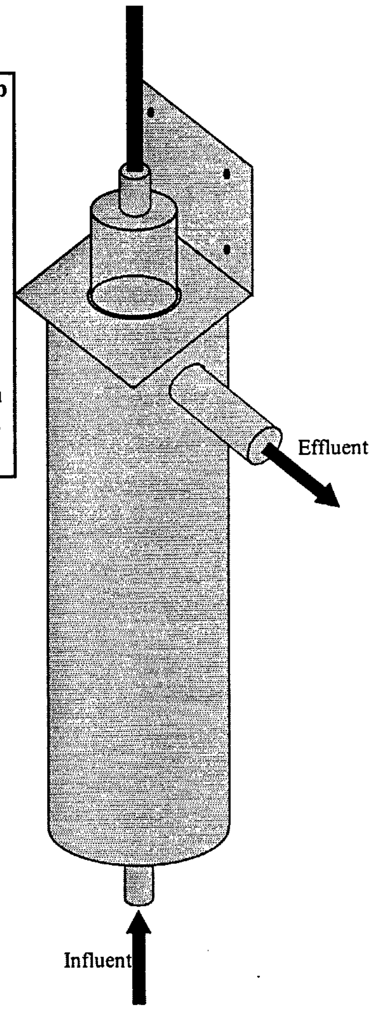
UV system and into the water collection tank. Figure 6 is a schematic showing the water path for initial loading of the filtration / disinfection process.

A single lamp system designed by Capitol Controls Group (Model JD-7 8101-JD) was the ultraviolet disinfection system used in this experiment. This system contained a single low-intensity mercury vapor lamp that was rated with a total lamp power of 20W with 5.3W of power emitted at 254.0nm. The lamp was encased in a stainless steel cylindrical shell with an inside diameter of 8.48 cm. Since the outside diameter of the

quartz sleeve was 2.45 cm and the lamp arc length was 27.1cm, a total reactor volume of 1.4L or 0.37Gal. was calculated. The manufacturer's rated flow was 7gpm, and the rated contact time was 3.18 sec. Additionally, the intended use for this system was to provide added disinfection to drinking water; and thus, the designed received dose for this system was 30mW-sec/cm^2 (assuming a percent transmittance of 95%). Figure 7, is a drawing of the UV system. A picture of the actual system can be seen in Figure 8.

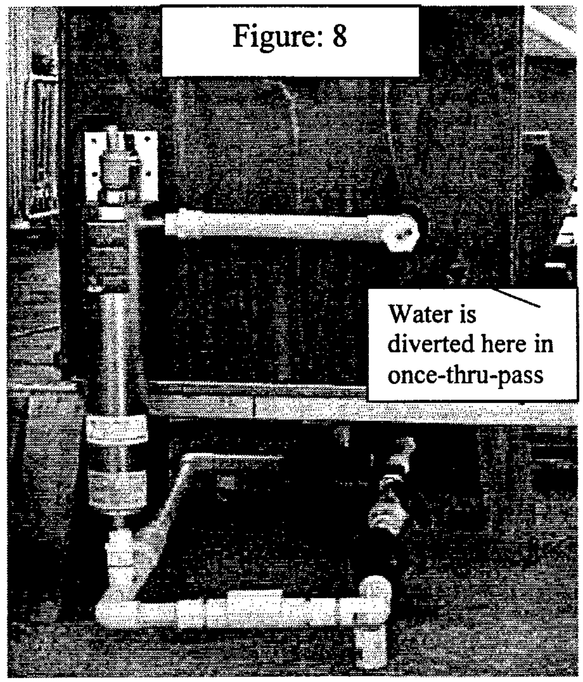
Figure: 7 Ultraviolet Disinfection System

Manufacturer: Capital Controls Group (Model JD-7 8101-JD)	
Lamp Power (Total):	20W
(@254nm):	5.3W
Lamp arc length:	27.1cm
Inside diameter of shell:	8.48cm
Outside diam. of quartz sleeve:	2.45cm
Reactor volume:	1.4L
Rated Flow:	7gpm
Rated contact time:	3.18sec.
Tested Flow:	~3.8 gpm
Tested contact time:	~5.84sec.



There are two distinct tests that were conducted to measure the effectiveness of the on-site UV system. The first test is the once-thru-pass. Figure 9 shows the pathway for the water in the once-thru-pass. In this test, the centrifugal pump located on the bottom line of the water collection tank pumped water from the collection tank through the UV system. Then the water was diverted to an additional collection container before entering the top of the collection tank again. This configuration ensured that the effluent

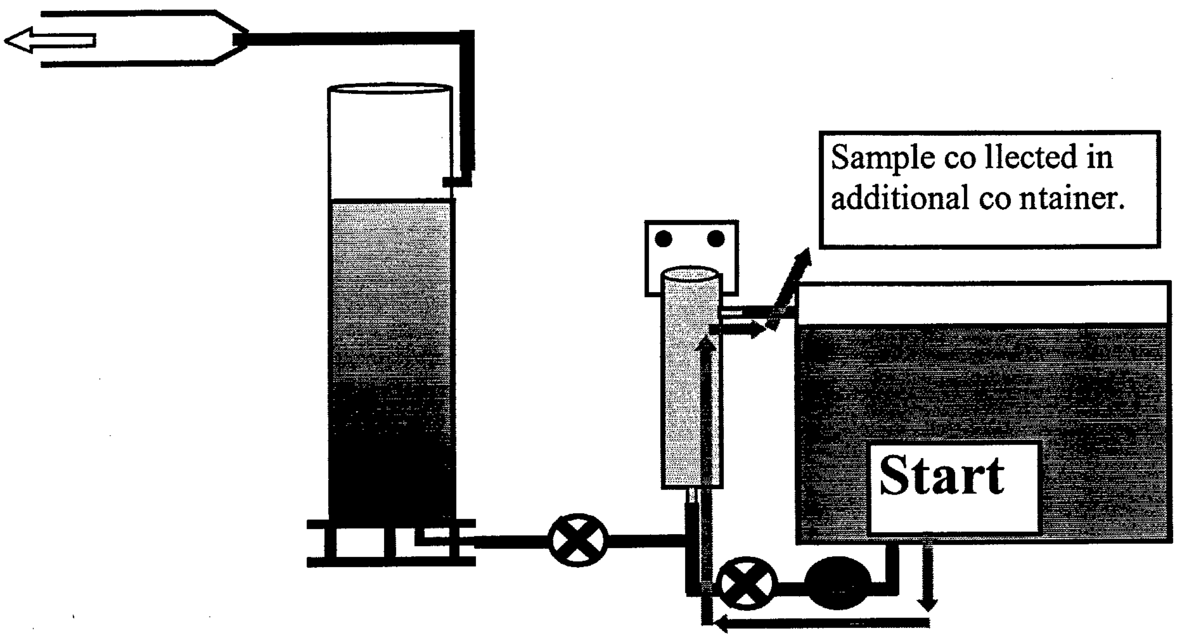
water sample received only one exposure to the light source before being testing in the laboratory. Figure 8 shows where the water was diverted before returning to the collection tank. Normally, for this test, either 20 or 40 liters was passed through the UV system before the test sample was taken.



The second test is the multiple-pass-test. This test recycles the water through the UV system for various numbers of times; and therefore, exposes the water sample to multiple doses of light from the UV system. Figure 10 shows the path for this test. Like the first test, the centrifugal pump first sent water through the UV system; however, the water was then returned to the collection tank and thoroughly mixed before it was passed through the UV system again. The total number of passes that the water makes through the UV system was determined by the flow rate of the pump. In the current configuration, the pump can generate a flow of approximately 3.8 gallons per minute. Therefore, if the total volume of water in the collection tank is known, the total number of passes can be determined by timing the entire process. Usually for this test, 40 liters (10.6 gallons) was recycled. Also, the test sample was not taken until the full time period for the passes was complete.

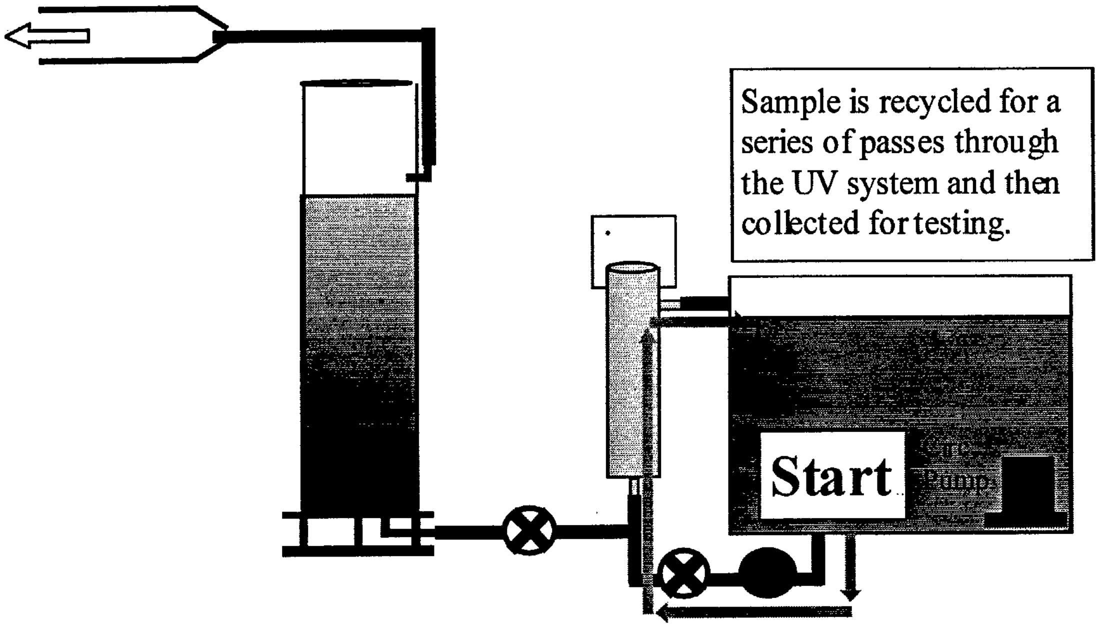
Two check valves are also shown in the schematic of the filtration / disinfection process (Figure 6) . The first check valve located between the sand filter and the UV

Figure: 9,Once-Thru-Pass



disinfection unit prevents water from flowing back into the sand filter during either of the two test runs. The second check valve is located between the UV disinfection system and the centrifugal pump. This check valve prevents flow to the centrifugal pump and the bottom feed line of the holding tank during the initial loading process. The use of these two check valves enabled the system to be designed with only one run of PVC piping. Otherwise, one complete line would be needed for the initial loading process and one line would be needed for the testing of the UV system. Additionally, the ultimate design of disinfecting the water continuously would be compromised. This idea for full-scale design will be discussed in further detail in a later section.

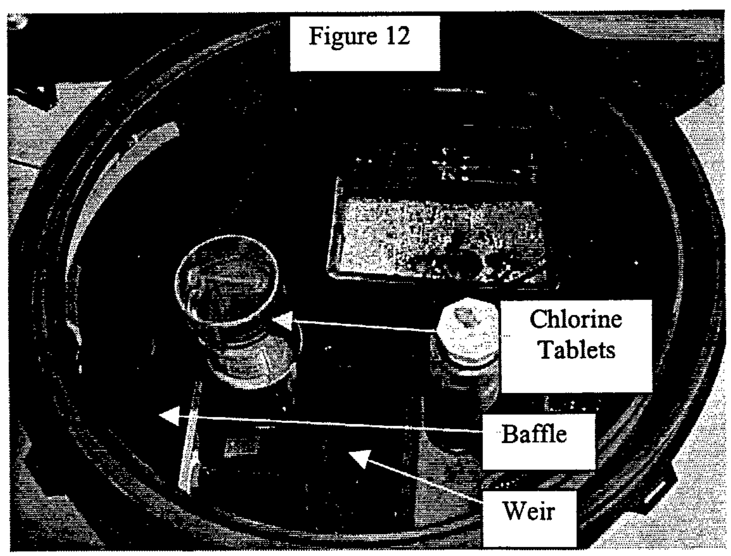
Figure: 10, Multiple-Pass-Test



Near the end of the testing period, sand filter performance was augmented with the addition of aluminum sulfate prior to filtering. This coagulation / flocculation process was performed by detaching the 1" PVC incoming pipe to the top of the sand filter and directing the secondary effluent into 5 gallons buckets. Aluminum sulfate (alum) was then added to each of the buckets in the appropriate dosage, stirred rapidly for 2 minutes, slowly for 15 minutes, and then allowed to settle for 20 minutes. Following 20 minutes of settling, the wastewater supernatant was then pumped into the top of the sand filter and then testing according to the particular experiment being performed (Figure 11).


Figure 11

The final testing performed in this series of experiments determined the effectiveness of the chlorine tablet disinfection chamber manufactured inside the WTPU. Until this point, this chamber was not used for disinfection in order to determine the effectiveness of the constructed filtration / disinfection processes described above. The volume of the chlorine contact disinfection chamber located inside the WTPU was 5.8 gallons. The chamber was constructed with a small weir that allows overflow into this chamber from the preceding sections of the WTPU (Figure 12). The weir directed all flow through approximately a three-inch wide channel. In the center of the channel was a perforated chlorine tablet canister. The canister rested inside the three-inch channel where all the overflow water was diverted from the weir. Then, when the canister was filled with chlorine tablets, mixing occurred between the water and the chlorine. Finally, there was one baffle that divided the last chamber into two sections. The baffle was approximately 12 inches in depth and prevented the newly dosed water from leaving the final chamber before the older water had left. The final chamber had a chlorine contact time that was completely dependent upon the total flow that was received by this system.



3.3 Analytical Procedures:

The following testing procedures were used to measure the performance of the system constructed:

<u>Standard Method # 9222D. Fecal Coliform Membrane Filter Test</u>

<u>Sample Collection:</u>

1. Collect sample aseptically using a sterile collection bottle (500mL).
2. Place sample in cooler with ice until ready for use.
3. Perform test procedure within 6hrs of collection.

<u>Equipment:</u>

The following equipment was used during this experiment:

1. Membrane Filters (0.45μm)
2. Tweezers
3. Isopropyl Alcohol
4. Bunsen Burner
5. Sterile 500mL sample collection bottles
6. 100mL Dilution Bottles
7. Vacuum Filtration Apparatus
8. 10mL Pipettes (0.1mL gradations)
9. Agar Plates (with fecal coliform agar medium)
10. Whirl bags (for 24 hour incubation of agar plates)
11. Precision 260, Circulating Water Bath (for 24 hour incubation of agar plates)

<u>Procedure:</u>

1. Filtration:

a) Select proper dilutions of sample to filter that will allow a yield count of fecal coliform colonies per membrane in the range of 20 and 80 colonies.

b) Prepare sterile filtration unit. With tweezers, place membrane filter over receptacle (grid side up) and lock in place.

c) Turn vacuum on. Filter 25-mL of sample through filter. Rinse through with sterile dilution water.

d) Turn off vacuum. Using tweezers, place filter in a culture dish.

e) Incubate filter for 24 hours at 45°C.

2. Counting

a) After incubation, remove samples.

b) Count the colonies blue in color. This represents the colonies produced from fecal coliform bacteria.

c) Compute the original count of fecal coliform using equation 1 and record results.

Equations:

The following equations were used during this experiment.

1. Fecal Coliform colonies/100mL = $\dfrac{\text{Coliform Colonies counted X 100}}{\text{mL sample filtered}}$

*(This equation determines the # of colonies actually filtered, if a serial dilution is used, the value must be multiplied by the inverse of the filtered dilution).

Standard Method #2540D - Total Suspended Solids

Equipment:

The following equipment was used during this experiment:

1. Whatman glass fiber filters (934-AH circles)
2. Aluminum weigh boats
3. 500ml graduated cylinder (5 ml gradations)
4. Vacuum filtration apparatus
5. Vacuum pump
6. Mettler AE200 Analytical Balance (accurate to 0.0001 g)
7. Thelco Model 18 oven (constantly set to 103°C)

Procedure:

1. A new filter (that has been stored in a dessicator) is placed in an aluminum weigh boat and weighed in grams on an analytical balance. The weight of the filter and weigh boat is recorded for later use.
2. The collected sample is measured to the nearest 1mL using a 100mL graduated cylinder. The volume of the sample is recorded for later use.
3. The filter is placed into the vacuum filtration apparatus (above the metal screen) and the sample is poured though the filtration system so that the entire sample passes through the inserted filter.
4. Approximately 10mL of distilled water are used to rinse the sides of the glass filtration apparatus and the graduated cylinder so that all suspended solids in the sample are on the filter.
5. The filter is then removed from the vacuum filtration apparatus, returned to the weigh boat and placed in an oven set at a constant temperature of 103°C for a minimum of 24 hours.

6. After drying is complete, the filter and weigh boat is removed from the oven and allowed to cool in a dessicator for a minimum of 10 minutes.
7. Finally, the filter and weigh boat is weighed in grams using the sample analytical balance in step 1.
8. Using equation 1 in the following section, the TSS concentration can be determined.

Equations:

The following equations were used during this experiment.

Total Suspended Solids: TSS (mg/L) = $(A-B)*10^6 / V$

Where: A = Final wt. of filter after drying in 103°C oven (g)

B = Initial wt. of filter (g)

V = Volume of sample filtered (mL)

Standard Method #2540 E - Volatile Suspended Solids

Procedure:

1. Using the same filter and weigh boat from Method #2540D, place the filter and weigh boat in the 550°C oven for 20 minutes after the TSS has been recorded.
2. After 20 minutes, the filter and weigh boat is removed from the 550°C oven and allowed to cool in a dessicator for a minimum of 10 minutes.
3. The filter and weigh boat is then weighed in grams using the same sample analytical balance used in determining the TSS.
4. Using the equation in the previous section, the VSS concentration can be determined.

Standard Method # 5210 B – 5 Day Biochemical Oxygen Demand

Equipment:

The following equipment was used during this experiment:

1. BOD bottle - 300 ml.
2. DO meter - YSI model 58.
3. Incubator - Equatherm.
4. Graduated cylinder - 100ml with 1 ml gradations.
5. Pipette - 10 ml with 0.1 ml gradations.
6. Pipettor.

Procedure:

1. Measured amount of the sample is poured into the BOD bottle.
2. To seed blank and glucose/glutamic acid standard 2ml of secondary effluent was added as seed culture.
3. Dilution water is added to all the samples till the BOD bottle is full up to the neck.
4. The probe of the DO meter is immersed in the bottle and the DO reading is recorded.
5. The BOD bottle is filled to the top with dilution water, stoppered and wrapped with film. Spillage is avoided.
6. The samples are placed in 20°C incubator for 5 days.
7. The DO in the samples is again read after 5 days with the DO meter.
8. Readings are recorded.

Equations:

$BOD_5 = [(DO_i - DO_5) - (DO_{si} - DO_{s5})f]/P$

where P = ml of sample/300.

f = ml of seed in sample/ml of seed in the seed blank.

DO_i = initial DO.

DO_5 = DO after 5 days.

DO_{si} = DO in the seed initially.

DO_{s5} = DO in the seed after 5 days.

Criteria: 1. Δ DO >= 2 mg/l

2. DO_5 >= 1 mg/l

Standard Method # 2130B. Turbidity, Nephelometric Method

Equipment:

The following equipment was used during this experiment:

1. Turbidimeter (Hach model 2100A)
2. Turbidity Sample Cell

Procedure:

1. Select an appropriate sensitivity range for sample. Adjust meter before each sample reading to the relevant sensitivity range.
2. Pour sample into sample cell.
3. Wipe sample cell free of fingerprints, dust or condensation.
4. Place cell into the meter.
5. Record turbidity.

Transmittance

Equipment:

The following equipment was used during this experiment:

1. Hach DR4000 Spectrophotometer
2. Quartz Cuvat

Procedure:

1. Set spectophotometer to wavelength: 254.0nm.

2. Set spectrophotometer to display results in percent transmittance "%T"
3. Fill quartz cuvat with distilled water, place in cell holder and press "Zero". This will calibrate the spectrophotometer so that the distilled water sample will display a reading of 100% transmittance (0.0 absorbance).
4. Remove quartz cuvat, empty and fill with unknown water sample. Place cuvat in cell holder and record result.

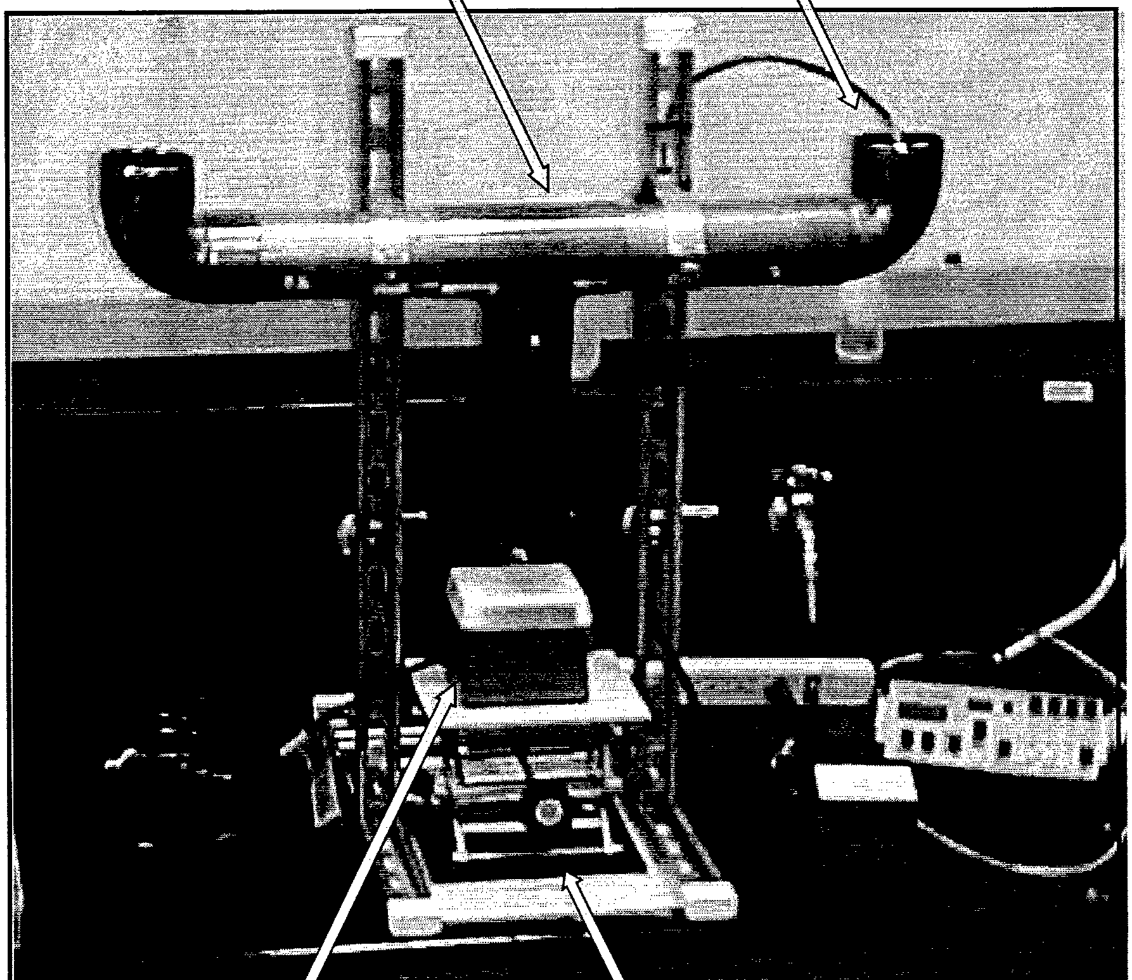
Collimated Beam Testing

Equipment:

The following equipment was used during this experiment:

1. Collimated Beam Unit (See Figure 13)
2. Corning Magnetic Stirring stand (See Figure 13)
3. International Light 1700, Radiometer (See Figure 14)
4. 50mL Petri Dishes (See Figure 15)
5. Magnetic Stir Bars
6. 50mL Graduated Conical Cylinder (varying gradations)
7. Hach DR4000 Spectrophotometer

Figure 13


3" diam. Copper pipe
Electrical wire to power source.
Magnetic stirrer
Laboratory Jacks

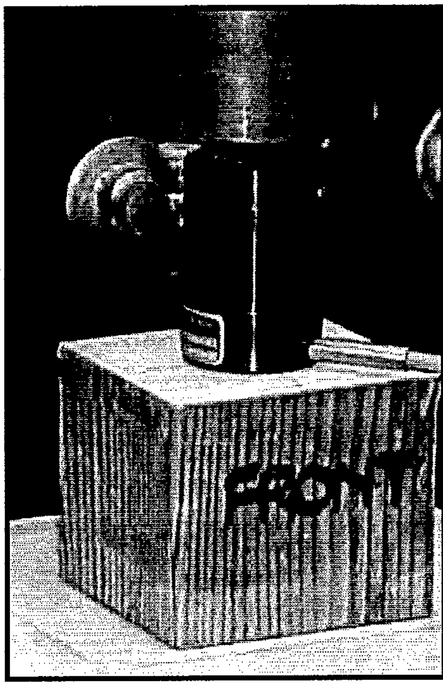


Figure 14:
International Light 1700, Radiometer (Measuring Intensity of the UV lamp before performing the collimated beam test)

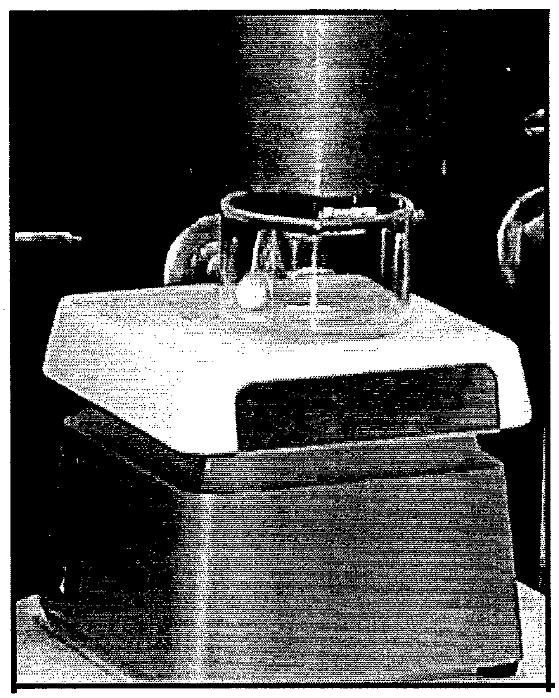


Figure 15:
50 mL Petri Dish
(In position for receiving UV Dose)
*In all cases, sample size is 50 mL.

Procedure:

The first step in performing a collimated beam test on a wastewater sample was to collect and prepare the sample. Each time a sample was collected, it was collected in a sterile 500mL plastic collection bottle. The samples were transferred from the site location (Sand Island Wastewater Treatment Facility) to the laboratory via a 15liter plastic cooler containing two cold packs. In all cases the experimentation was completed within 6 hours of collection. Also, in all cases the wastewater samples remained in the cooler until it was needed for examination in the experiment.

The next step was to determine the percent transmittance of the wastewater sample. Using the Hach DR4000 spectrophotometer, distilled water was placed in a quartz cuvat and inserted into the spectrophotometer to zero (or adjust) the spectrophotometer to a transmittance of 100%. Then the quartz cuvat was filled with the wastewater sample and measured for transmittance.

Following the determination of the percent transmittance, the determination of exposure time to the collimated beam unit needed to be calculated. For all tests in this experiment, collimated beam unit number 2 at the University of Hawaiʻi Environmental Lab was used to provided consistent data between collimated beam tests. First, the unit was measured for intensity at a given distance from the collimating tube (see Figure 14). Distances that yielded even numbers on the radiometer were used to make calculations easier. A distance corresponding to an intensity of 240mW/cm^2 was used for this unit at each experiment. Using a correction factor to correct for the absorbance of the light in the water sample, an average intensity throughout the depth of the 50mL water sample could be determined. The correction factor used was (Morowitz, 1950):

$$C = (1 - e^{-\alpha L}) / \alpha L$$

Where: C = correction factor

α = Absorbance (/cm)

L = Sample depth (cm)

The sample depth of each 50mL petri dish was 2.2 cm. Finally, exposure times were calculated to obtain doses of 5, 10, 20, 30, 40, 50 and 75 mW-sec/cm^2.

To expose the 50mL samples of effluent to the UV source, the following procedure was used. First, the sample was measured to 50mL using a conical cylinder and then poured into a 50mL petri dish. Then, while placing a cover (a cardboard plate) over the dish, the dish was inserted underneath the collimating tube on the collimating beam unit (see Figure 15). After a timer had been set for the exposure time for the tested sample, the cardboard plate was removed and the timer was started. After the exposure time was complete, the cardboard plate was returned to cover the petri dish from the UV source, and the petri was then removed from the collimated beam unit stand. The sample was then ready for analysis using the membrane filter technique to determine the number of reproducing fecal coliforms remaining in the sample.

Chapter 4
Results

4.1 Effectiveness of the Wastewater Treatment Package Unit

The wastewater treatment package unit used in this project provided a varied quality of secondary effluent with which to begin the filtration and disinfection processes. The WTPU (which has a total capacity of approximately 800 gallons) received a daily flow of 400 gallons per day. This yielded a 2-day retention time. The daily loading of 400gpd was determined under the assumption that this unit would service a four-person household with each individual contributing 100 gallons of wastewater per day (Metcalf & Eddy, 1991; NSF Standard 40). This is different from the manufacturer's rated capacity, which intended this unit to treat domestic wastewater for a family of five in Japan. In Japan, however, the average per capita flow for domestic wastewater is approximately 66 gallons per person per day. Therefore, the manufacturer's maximum hydraulic loading capacity would be approximately 330gpd. Thus, the tested loading of 400gpd is overloading the WTPU by approximately 21% from its designed maximum daily hydraulic capacity. From this analysis it can be assumed that the system was tested at its maximum hydraulic capacity; and that the effluent quality tested is likely the worst case scenario for this tank.

There are other factors that must be considered, however, when determining the quality of the effluent tested in this experiment versus the quality of the effluent from a residential unit. First, since the raw wastewater used for this testing was obtained from Sand Island Wastewater Treatment Plant (SIWTP), the water characteristics are different than wastewater from a home. One of the primary ways in which SIWTP's untreated

wastewater differs is that there is a significant amount of saltwater intrusion into the system. Depending upon the time of day, salt water intrusion accounts for 15 - 45 percent of the total inflow into the plant (Salt Water intrusion data provided by SIWTP). This is radically different from a home use situation which would have zero saltwater intrusion in most cases. Additionally, wastewater that enters SIWTP comes from industrial sources as well as residential areas. This also changes the quality of the water and may include chemicals which normally would not be found in wastewater from a residential unit. Secondly, another major factor that might cause the quality of the effluent in this test to vary from a real (single family home) situation is the loading schedule that was used for this testing. The loading criterion used to fill the tank with 400gpd of raw wastewater followed the protocol described in the National Sanitation Foundation (NSF), Standard 40 for the production of class 1 effluent. This schedule consisted of following the periods shown below.

TABLE 1 – WTPU LOADING SCHEDULE

TIME OF DAY (HRS)	% OF DAILY FLOW	VOLUME (GAL)
0600-0900	35	140
1100-1400	25	100
1700-2000	40	160
Totals	**100**	**400**

This schedule loaded the tank at three time periods per day. Although this loading schedule may simulate an average household flow, it does not account for exceptional circumstances such as a heavy laundry day, vacation periods, spring cleaning

etc. The NSF Standard 40 protocol does require testing circumstances where this typical loading schedule would not be followed (i.e. stress tests); however, these tests outlined in the standard were not performed during the testing period to create water of reusable quality.

Finally, the quality of water tested at the SIWTP rarely contained particles of 5mm in diameter. This result would be far different than if the test was performed at a residential unit where the wastewater did not have the time to partially solubilize the larger particles. Thus, the microbial growth rates of the free and fixed microorganisms in the tank chambers would be altered. Therefore it is difficult to know the effectiveness of this experiment with regards to the conditions the WTPU would receive when operating at a residential unit.

It is worthy to note that although the testing performed on the WTPU was extensive, it did not determine if the testing performed taxed the tank to its maximum limits. Based on some of the reasons discussed, it is likely that the tank was tested more at typical operational levels. Also, it is difficult to say whether the wastewater tested was of similar quality to that of a single household.

Perhaps for the purposes of this study, it is best to simply compare BOD_5 levels of SIWTP untreated wastewater with the expected levels at a typical, four-person, US family home. From our study the average BOD_5 level that entered this tank was 146 mg/L. When this value is compared with typical values for BOD_5 in a four-person home, however, this is lower that typical values. Using unit waste loading factors originally published by the Water Pollution Control Federation, BOD_5 ranges between 0.13 and 0.26 pounds per capita per day (Metcalf and Eddy, 1991). This is a range of

approximately 155 – 312 mg/L, assuming that a four-person family produces 400gpd of wastewater, which was the case for the hydraulic loading of the WTPU. Average values for BOD5 range between 215 – 265 mg/L depending upon if kitchen disposal wastes are included in the wastewater.

Table 2 shows the typical range for suspended solids using the unit waste loading factors and a loading of 400gpd at a four-person home.

TABLE 2 – MEASURED VS. TYPICAL PARAMETERS

Parameters	This Study	Typical domestic values*
BOD_5 Influent (mg/L)	146	155-312
Total Suspended Solids (mg/L)	128	155-395

*Metcalf & Eddy,1991

As this table shows, the levels measured during this study are closer to the lower values stated for typical domestic wastewater. Therefore, this study may not be indicative of the concentrations possible in a single family home. Clearly, the next step of this study would be to connect the WTPU to a single family home and test the results.

Regardless of the variations between the wastewater tested and the expected values in a home use situation, the WTPU did achieve effluent suitable for this study. The effluent from the tank averaged at 14 mg/L for BOD_5, 11 mg/L for total suspended solids and 8 mg/L for volatile suspended solids. Additionally, the average turbidity from the six months of testing was 5.6 NTU (Figure 16), and the average percent transmittance was measured at 58.8% during this period (Figure 17).

These values present some challenges for reuse. First, note that the wastewater effluent does not meet the secondary effluent standards for turbidity prior to direct

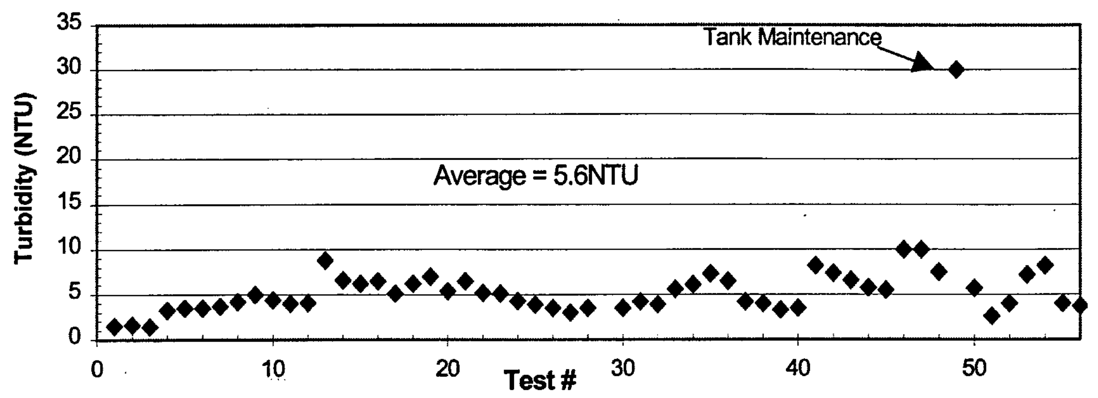


Figure: 16 Effluent Turbidity (unfiltered)

filtration. The state DOH guidelines clearly state that any secondary effluent that is over 5NTU must be chemically coagulated before filtration or diverted to other than R-1 reclaimed use. Of the times that turbidity was recorded over the six-month testing period, the turbidity exceeded 5 NTU fifty percent of the time. Therefore, if this regulation were

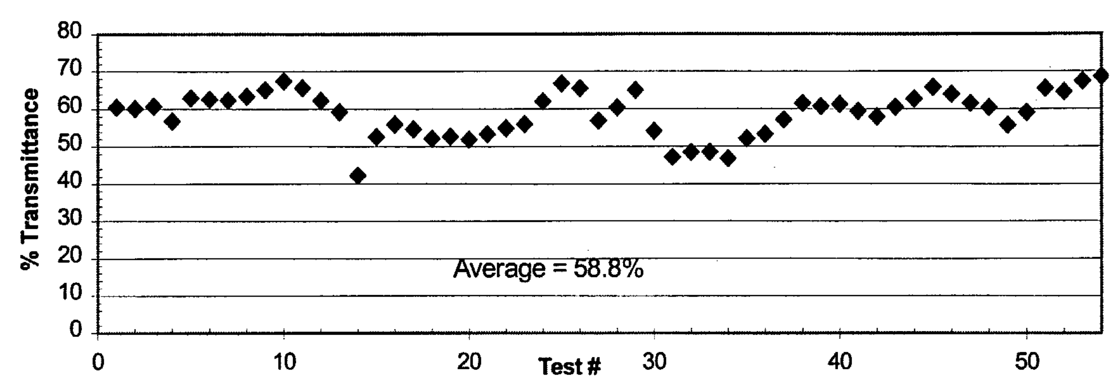


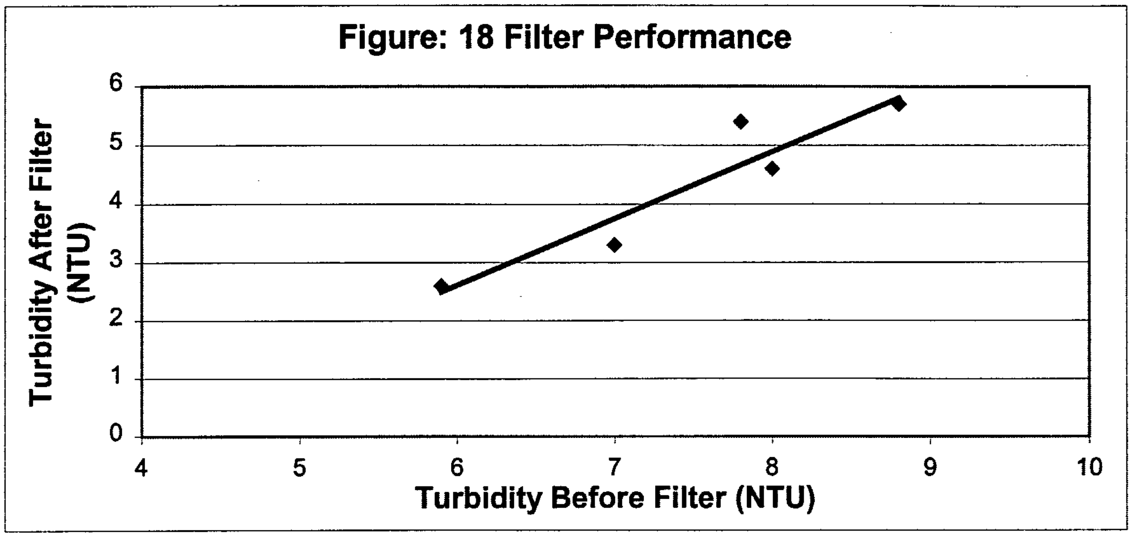
Figure: 17 Effluent Transmittance (filtered)

strictly enforced, an automatic chemical addition system would be required. This is considered impractical. Another problem with the initial quality of the water is the effluent transmittance. The DOH guidelines require the wastewater to be above 55% after filtration. This could be a roadblock to reclaimed water production because the results from Figure 17 show that the transmittance after filtration (using a laboratory

glass fiber filter) drops below 55% approximately 27 percent of the time. This occurrence happens at the same time the turbidity rises above 5 NTU in all cases.

4.2 Results of Sand Filtration Testing

Since this experiment was not subject to the DOH guidelines, (because all wastewater was diverted back to the raw wastewater channel at SIWTP) wastewater



above 5NTU and below 55% transmittance was sent directly through the sand filter without prior chemical coagulant addition. As explained in the methods section of this report, a dual-medium sand filter was initially used to filter the water. Turbidity measurements were taken before the sand filter (collected in the last chamber of the WTPU) and after the sand filter (collected in the holding tank). Based on the before and after turbidity measurements, Figure 18 was created. This shows that from our testing an improvement of approximately 44% was achieved. This is a relatively low percent improvement compared with 70% which is possible by some sand filters (Asano, 1998). Additionally, the data used in this graph included only five tests run between 3/22/99 and 4/2/99. Earlier testing proved to be inadequate since the sand filter was not initially backflushed before testing; and the later testing showed that continual deterioration of the effluent turbidity was apparent. It is expected that the continual deterioration of effluent

turbidity was due to dirty site conditions and a poorly designed gravel bed. The mistakes made and lessons learned will be discussed at the end of this section.

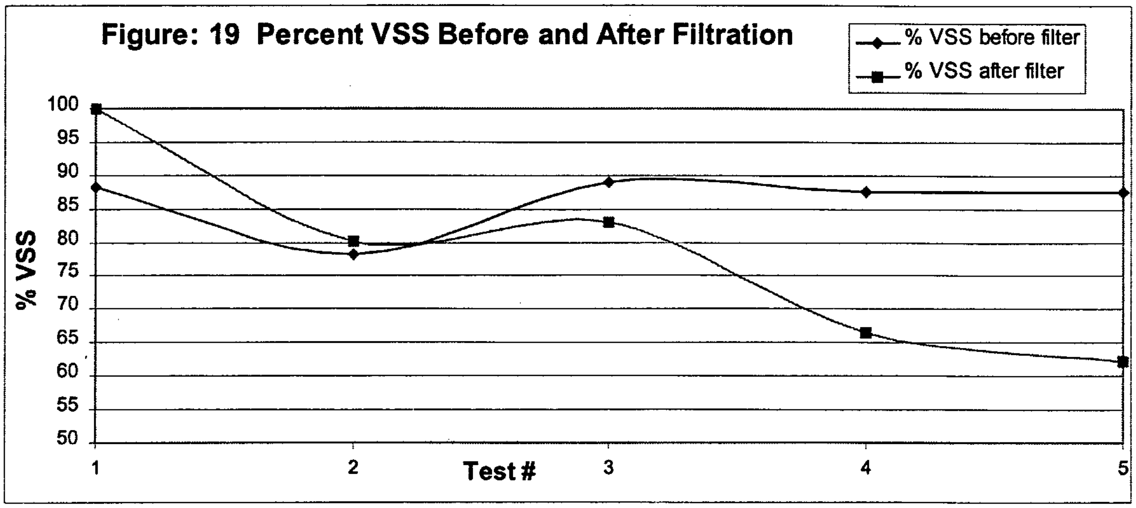
If at this point, the maximum initial turbidity possible to obtain 2NTU after sand filtration (using the current percent improvement of 44%) is calculated, the secondary effluent could not exceed about 3.6NTU. This is even lower than the regulated criteria of 5NTU and would limit the possible times that R-1 quality water could be produced even further. Fortunately, better sand filters are available and could be implemented in this system. If a sand filter capable of removing 70% of the turbidity was used, this would raise the maximum initial turbidity of the secondary effluent to about 6.7NTU. This is a definite improvement, but it still does not account for the overall fluctuation in the secondary effluent produced by the WTPU. The fluctuations in Figure 16 showed that the turbidity exceeded 6.7NTU approximately 20% of the time. This is still a major limitation to consistent quality water. However, since this limitation already assumes the best possible removal from a sand filter, it is unlikely that turbidity levels will be low enough to pass current DOH criteria using sand filtration alone. Clearly an additional process would be needed to ensure turbidity levels below 2NTU on a consistent basis.

Many mistakes were made in the testing process; and consequently, much was learned from the experimental process. As stated earlier, the design of the sand filter was less than ideal. The sand filter used was a dual-medium, essentially unstratified, rapid sand filter. After the first backwash on March 22, 1999 an improvement in the turbidity of 56% was recorded. However, the percent improvement in effluent turbidity rapidly decreased until it was only 31% after five tests. This is an extremely rapid deterioration of effluent quality and indicates poor filter design. Likely reasons for the poor filter

performance are based on two critical observations that were made during the testing. The first observation was that there was a significant amount of sand leaving the filter and clogging the piping of the system. In all cases, the sand that was escaping the filter was 0.1mm or 90-grit quartz sand. This indicated that the gravel bed below the sand layers was insufficiently designed to contain the smaller sand particles in the filter. Consequently, it is likely that suspended particles that cause turbidity could have been attached to the fine sand grains that escaped from the filter. Also, the loss of sand grains could lead to the possibility of preferred flow paths through the filter which would add to its inefficiency.

The second observation made during testing was that the fine sand grains used in this filter were below the typical values stated in all researched literature (Asano, 1998; Metcalf & Eddy, 1991; Vigneswaran et al., 1995) with 0.2mm grains being the smallest recommended size. It is possible that the smaller grain size reduced the efficiency of the sand filter due to the increase in flow velocity between the sand grains. Consequently, high velocities between grains within the filter medium inhibited filtration mechanisms such as impaction, adhesion, sedimentation and interception. It is also possible that the sand grains themselves could have been the cause for increased turbidity if they had remained in suspension after escaping the filter. This may especially be true since the 0.1mm sand used did not have a manufacturer's uniformity coefficient, indicating that some of the particles could have been much smaller. One indication that this may be the cause of inefficient filtration can be seen in a graph of the percent of volatile suspended solids remaining after filtration. This graph (Figure 19) shows that while the % VSS before the filter remained fairly constant throughout the tests, the % VSS after filtration

slowly decreased. This means that there was an increase in % fixed solids through the filter. Furthermore, the direct measurements show that the percent increase is due to an



actual concentration increase in fixed solids. Thus, the filter actually added to the concentration of fixed solids in the filtered effluent. This observation lends credence to the assumption that the smaller sand grains may actually be the cause of increased turbidity in the effluent.

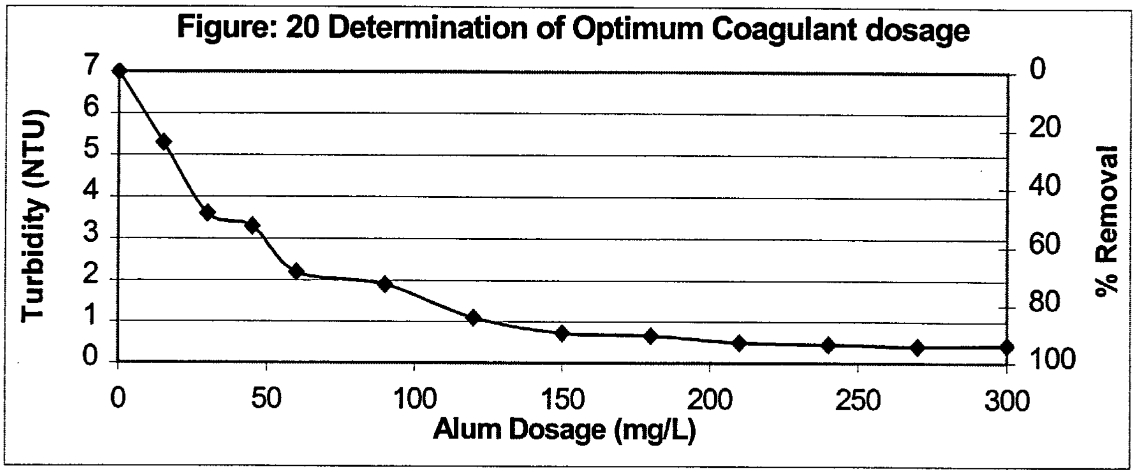
In addition to the dual-medium sand filter used in this experiment, a mono-medium sand filter was also tested near the end of the experiment. This filter was built with the intent of correcting many of the problems encountered in the first filter. The mono-medium filter used 30-grit quartz sand (0.5mm) with a bed depth of 12 inches. Also, a highly porous, heavy screen fabric was placed between the gravel bed and the sand bed to prevent excessive mixing of the sand into the gravel layer. Immediately before use, this filter was backwashed to clean the sand layer of any small particles attached to the sand. In addition, the filter was filled with clean water and analyzed for the loss of sand particles through the filter. This time the filter performed without any

loss of sand. Unfortunately, the fouling of the entire system prevented accurate testing of this filter; thus, no additional data was obtained using the improved filter.

It was noticed at this time that the entire system had become filled with the smaller sand particles. Small deposits of sand were noticed in the piping immediately exiting the sand filter and in the piping after the UV disinfection reactor. The intrusion of sand throughout the system prevented any further testing of the old and new filter since any sample taken after the filter showed little or no improvement in effluent quality. Additionally, the current system design prevented any means of collecting sand (i.e.: a sand trap) before complete spreading throughout the system. Also, this system did not provide a means for cleaning out the piping without completely disassembling the system. The construction of the system and possible improvements will be discussed in further detail in the discussion section of this report.

Since sand filtration alone could not provide the type of removal necessary to consistently reduce the turbidity levels produced from the WTPU, coagulation, and flocculation processes were tested. Approximately 10 gallons of the secondary effluent wastewater was brought back to the lab for jar testing in order to determine an optimum dose for effective coagulation and flocculation. Aluminum Sulfate was the chemical agent used for this experiment to provide coagulation and flocculation prior to the sand filter. Jar testing was performed in the laboratory following recommended coagulation, flocculation, and sedimentation periods outlined in Eckenfelder and Ford (1970), and explained in the system construction section of this report. Figure 20 shows the results of the laboratory jar testing. This graph shows varying doses of alum ranging from 15mg/L to 300mg/L. From this test it is apparent that improved turbidity continues until

approximately 200mg/L of Alum was added. At this point, increased doses of Alum provide almost no additional settling or removal of suspended particles in the water. Therefore, 200mg/L was chosen as the optimum amount of chemical coagulant to use for this testing. In an actual OWRS, 200mg/L of alum would not be required since the



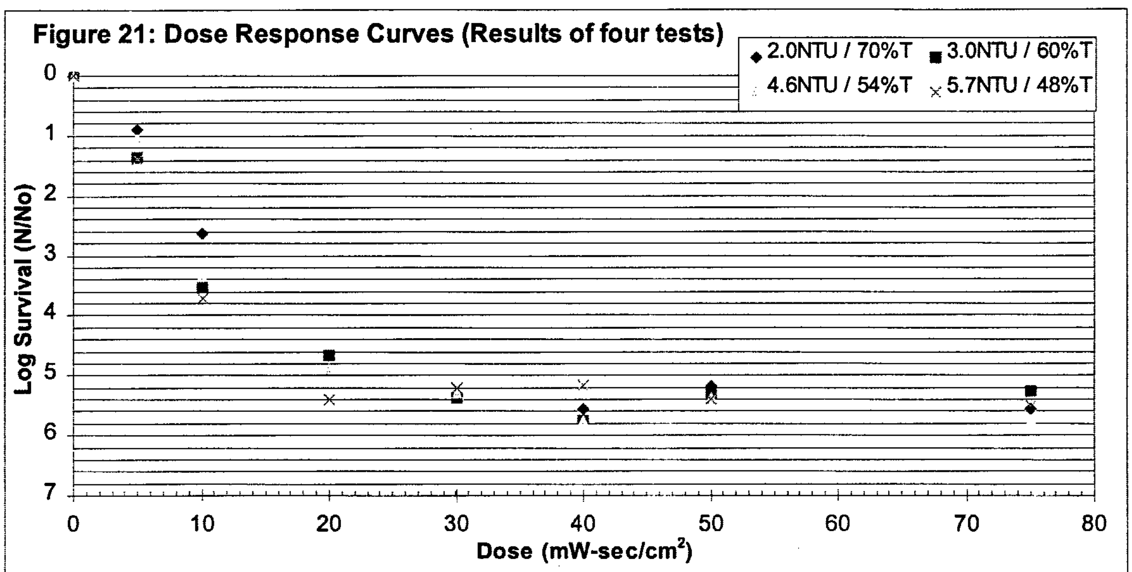
effluent prior to filtration only needs to be below 5NTU. A high alum dose was chosen in this test for the purpose of proving the effectiveness of the process. In an actual OWRS, however, the highest percent removal required would be about 50% since the guidelines require that any secondary effluent above 10NTU not be used for R-1 water. Thus, 50% removal from the coagulation / flocculation process would lower secondary effluent from 10NTU to 5NTU. Figure 20 shows that greater than 50% removal can be achieved with only an alum dose of 50mg/L. 50mg/L would probably be a more likely alum dose in an actual onsite water reclamation system.

Based on the field results, 84% removal was achieved on site producing water with a turbidity of 1.7NTU prior to filtration. The % removal was a little lower than what was achieved in the laboratory, but this was due to the less than ideal conditions for coagulation and flocculation onsite. As explained in the system construction section,

five-gallon buckets were used to perform the coagulation and flocculation, and manual stirring was performed to aid the two processes. Thus, ideal mixing conditions were difficult to simulate onsite. After filtration using the new mono-medium sand filter, there was a noticeable rise in the overall turbidity of the sample. This was again due to the fouling that had occurred throughout the piping in the entire system. Therefore, the R-1 requirement of filtered water below 2NTU was not met. None-the-less, the coagulation/flocculation testing did show that with chemical assistance, the final turbidity requirement of less than 2NTU is possible.

4.3 Results of Ultraviolet Disinfection Testing

Now that filtered effluent had been obtained, a study on the effectiveness of the ultraviolet disinfection of the single lamp system could commence. A 500mL sample of filtered effluent was returned to the lab for collimated beam testing. Using the method for collimated beam testing explained earlier, four dose-response curves were generated. Figure 21 is a graph of the four collimated beam tests. As shown in the legend, the turbidity of the samples tested varied from 2NTU to 5.7NTU. Interestingly, the log removal from each test did not vary dramatically. The expected result should have been a noticeable decrease in the log removal as the turbidity increased. However, the result was quite the opposite in some cases. Perhaps the reason is based upon the observations

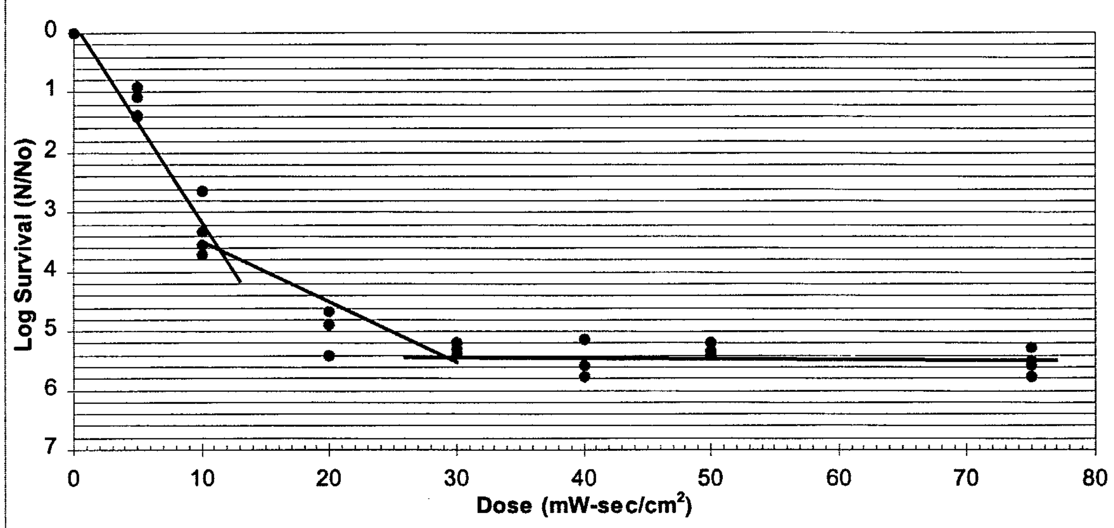


noted in the previous section. In the results of the sand filtration testing, it was suggested that a portion of the turbidity in the filtered effluent was due to fixed solids leaving the sand filter and becoming suspended in the effluent. If this were the case, these particles would have little effect on shielding the microorganisms from the ultraviolet light because the shielding effect that microorganisms receive from suspended particles must

be from all sides. Organic particles that contain bacteria inside the particle are ideal shields for bacteria, but fixed solids that enter the water hours before the collimated beam test is performed will provide little shielding. Therefore, it is possible that the noticeable changes in turbidity between the four samples would have little effect on the UV disinfection effectiveness in this experiment.

Once the four collimated beam tests had been performed, a general bioassay graph for fecal coliforms could be generated. Figure 22 is the expected dose-response curve for fecal coliforms when exposed to ultraviolet light. This graph was generated

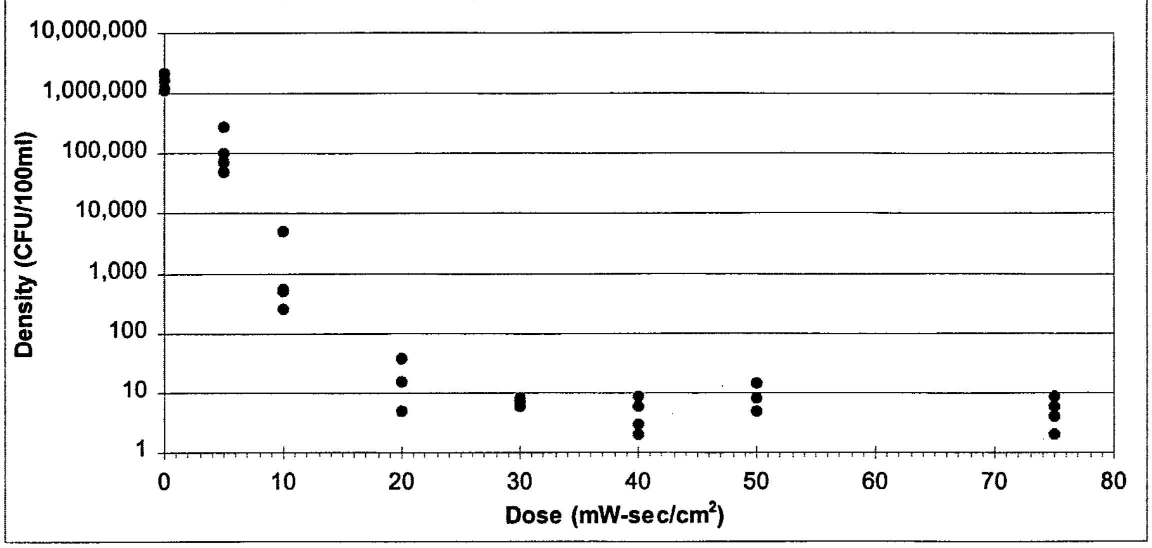
Figure 22. Dose Response Curve (Compilation of Test Results)



using linear regression to determine the best fit lines from the results of the four collimated beam tests performed. The three lines on this graph represent three changes in the effect that ultraviolet light has on fecal coliforms. The first line represents the kill rate of free microorganisms or those that lack any shielding from the ultraviolet light. In this section of the graph, a minor increase in the dose greatly affects the percent of the removed microorganisms. The last line represents the kill rate of shielded fecal coliforms. At this point in the test, an increase in the applied dose did not create any

change in the number of microorganisms removed. This can be seen more clearly in Figure 23, which is a density plot of the collimated beam tests performed. Note that very few organisms remain at the higher doses, yet these are fully shielded and practically

Figure 23: Density Plot of Collimated Beam Testing

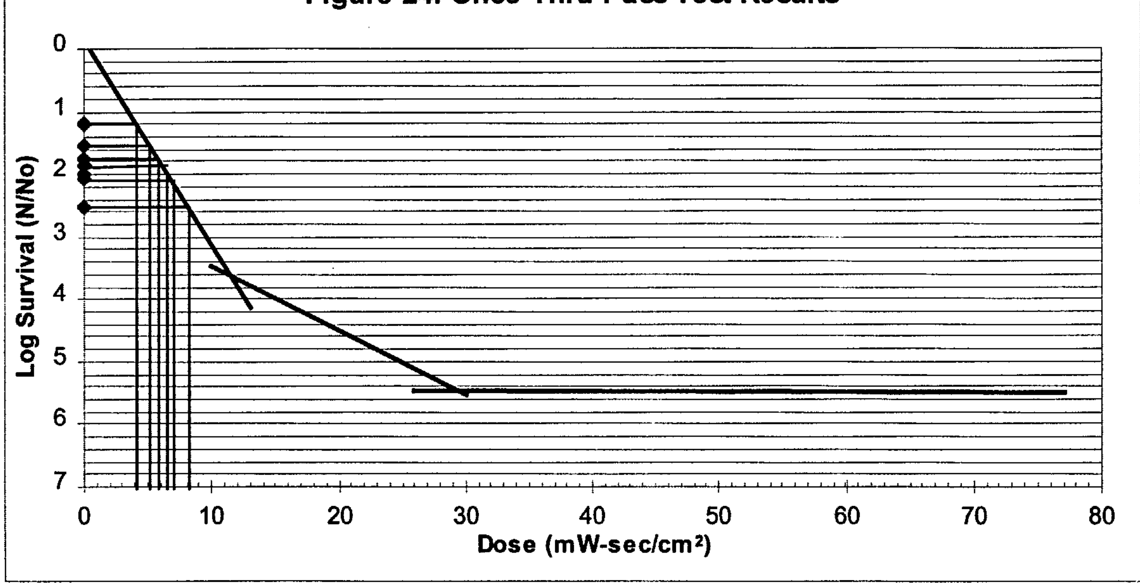


immune to the effects of the UV light. Also note that the remaining values are above the R-1 criteria of less than 2.2CFU/100mL which means that improved filtration is necessary to remove the particles shielding the remaining microorganisms. Finally, the middle line shows the transition between the free organisms and the shielded microorganisms. This section of the graph represents those fecal coliforms that are partially shielded and therefore require a higher dose of radiation to render inactive. Yet these microorganisms are still able to be affected by the UV light and ultimately succumb to the inactivation effects of radiation. A curve would more accurate represent this section since it is a transition zone between the first and last lines. Since this section is not used in the analysis, however, a line is suitable for this experiment.

As shown earlier in Figure 9, the filtered effluent that had been collected was pumped from the bottom of the collection container through a check valve and then through the UV reactor. For the once-thru-pass test, the water was collected in another container and then discarded back into the main waste stream at Sand Island Wastewater Treatment Facility. To ensure that a collected sample accurately reflected typical operating conditions, the sample was taken after at least 20 liters of water had passed through the UV system. This was done so that the flow through the UV reactor would reflect a steady state flow condition. Additionally, before the commencement of a UV test, the reactor was filled with water and the lamp was turned on for a minimum of five minutes to allow proper warm up time for the lamp.

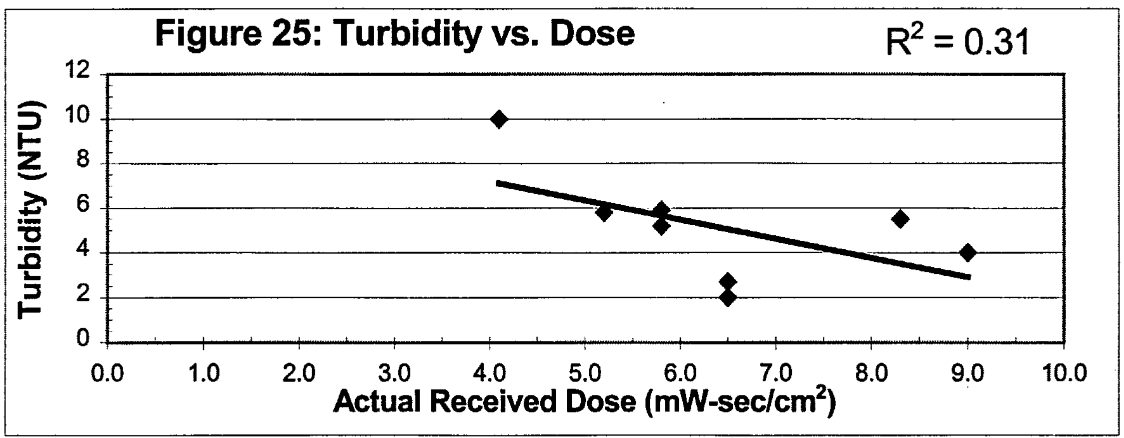
Following collection, the sample was returned to the laboratory and tested using the membrane filtration method to determine the effectiveness of the UV disinfection system. Figure 24 shows the average effectiveness of the single lamp system. As shown

Figure 24: Once-Thru-Pass Test Results



in this graph, the log survival from a once-thru-pass test varied from 1.5 to 2.5. Since the

DOH guidelines closely monitor turbidity, it could be assumed that the variance was due to turbidity as is shown in Figure 25. In the two figures, the higher doses generally relate to the lower turbidities and the lower doses generally relate to the higher turbidities, however, the tests performed did not always produce the expected results. For example, the lowest turbidity test of 2.0 NTU did not yield the highest log removal; and therefore, this test did not yield the highest dose. Rather, the highest dose was from a sample containing a turbidity of 4.0NTU. Also, the R^2 value for Figure 25 was only 0.31. This shows that turbidity was not a good indicator to determine the actual received dose. Clearly, factors other than turbidity affected the results of this test.



The variable population of microorganisms in the wastewater stream was the first factor evaluated for explaining the results. During the early stages of the disinfection testing period (3/9/99-3/11/99), it was apparent that the WTPU needed semi-annual maintenance since the turbidity levels became unusually high after 6 months of continuous operation. Therefore, to revive the tank, half of the total contents from the WTPU was pumped out. This cleaning varied the fecal coliform counts in the tank from an average of $1.2x10^6$ CFU/100mL to a high of $8X10^6$ and then to a low of about 100,000 CFU/100mL. It is suspected that this variation effected the log removal determined by the

membrane filtration technique when the overall population was in such a state of flux. An effective UV dose could have been miscalculated because a larger population could have provided shielding to a different number of microorganisms than a smaller population. Also, a decrease in the population of fecal coliforms could have affected the overall health of the population. In other words, the extent of the inactivation could have been overestimated due to the physical state of the microbes (Sobsey, 1989). In any case, the variation in the population of fecal coliforms did play a role in the variance of the log removal noticed and should be noted.

To minimize the effects of the fecal coliform population flux, all collimated beam tests were performed when the microbial population was near average levels. One collimated beam test was performed one month prior to the tank maintenance and the other three collimated beam tests were performed two weeks after the tank maintenance and when the system was operating within typical parameters again. Three of the eight once-thru-pass tests, however, were performed when microbial populations were lower than average and one microbial test was performed when the microbial population was near the high value in this range. All four tests performed when the fecal coliform population was unstable produced lower than average log survival values.

Other variations in the observed log removal of the once-thru-pass include the testing method used with the UV lamp. The 5-minute warm up period was arbitrarily chosen to give the lamp a short period to reach typical luminescent conditions. No actual warm up period was recommended by the manufacturer, therefore, the warm up period may not have been sufficient for the lamp to reach a steady state condition. Under ideal conditions (in an actual system) this would not have been a concern because the lamp

most likely would have remained in operation continuously; but during this testing process, the lamp could only be on at certain times. The lamp had to be turned off periodically because the water passing from the filter to the holding container during the initial loading could not be disinfected or else the once-thru-pass could not be determined. Also, the lamp was only fully submerged during the test and could not remain in use when system was emptied and unattended.

Not only was the variation in the values obtained from the once-thru-pass test noticeable, but the values were also much lower than expected. Based on the values graphed in Figures 24 and 25, one log survival was obtained at a received dose of 3.5mW-sec/cm^2 and the average received dose recorded from the once-thru-pass test was 6.4 mW-sec/cm^2. Additionally, the average received dose of 6.4mW-sec/cm^2 corresponded to a 2 log survival or a 99% removal of fecal coliforms. The average received dose from the once-thru-pass tests was dramatically lower than the 30mW-sec/cm^2 dose published by the manufacturer. However, the manufacturer's value was for a transmittance of 95% and so this comparison was really of no use since the transmittance of the filtered effluent averaged around 60%. The manufacturer of the UV lamp system was available for comment on a few occasions for discussion concerning this output. Based on a point source summation method used exclusively by Capitol Controls Group, the manufacturer estimated that a 16.6 mW-sec/cm^2 applied dose could be obtained for wastewater with a transmittance of 60%. Capitol Controls Group would not release the exact way in which they used the point source summation method to obtain this value, but it was probably similar to the general method explained in the theory section of this report. This value stated by Capitol Controls Group was also based

on a reactor flow rate of 7gpm or a contact time of 3.18sec as compared to the 3.8gpm flow rate used in this experiment which corresponds to a 5.84sec contact time.

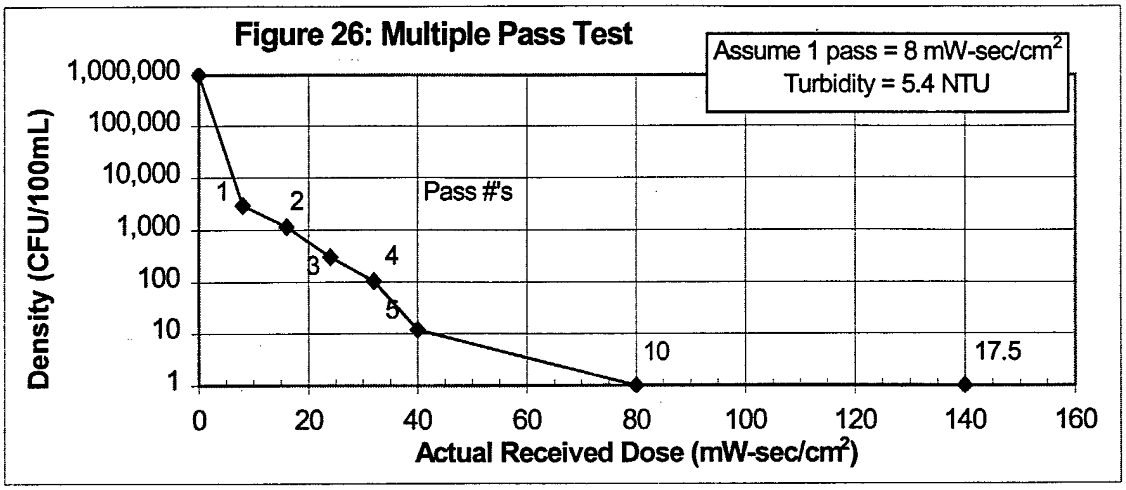
Thus, based on a longer contact time alone, the value obtained in the bioassay should yield a higher dose than the applied dose obtained from the manufacturer. Using simple geometry, an average intensity can be calculated by dividing the lamp power by the average contact area between the quartz sleeve and the reactor wall. Also the average intensity must be corrected for lamp age (factor of 0.7) and the transmittance through the quartz sleeve (factor of 0.7). This rough method to determine the average intensity yields a value of approximately $5.6mW/cm^2$ for this UV system. If this value is multiplied by the manufacturer's recommended contact time of 3.18 seconds, the applied dose is $17.7mW\text{-}sec/cm^2$, which is close to the point source summation value received from the manufacturer. When the contact time is increased to the time tested in the bioassay (5.84 seconds) an applied dose of $32.6mW\text{-}sec/cm^2$ is calculated. Comparing this value with the actual received dose determined in the bioassay, the reactor efficiency is determined. This experiment showed that the UV reactor had an efficiency of only 20%. This is a poor reactor efficiency and must be partially due to factors other than the reactor design.

It is suspected that the primary reason for the discrepancy between the manufacturer's applied dose and the actual received dose determined from the bioassay is the condition of the UV lamp's quartz sleeve. Many resources have stated that the key element of an effective UV system is the cleanliness of the quartz sleeve (EPA, 1992b; White, 1999). This concern was not considered during the testing because the entire testing period lasted for about a month which is about the usual time period that a lamp is cleaned (EPA, 1992b). Therefore, fouling of the quartz sleeve was not expected during

the testing period. After the testing period, the quartz sleeve was removed and although the outer surface of the quartz sleeve was still clean, the inner surface was coated with moisture. This was an unexpected scenario since the inner surface of the quartz sleeve was not in contact with the wastewater passing through the reactor. The most likely cause of this buildup of moisture inside the quartz sleeve was infiltration of moisture from the outside air. The system design was made to keep the lamp fairly protected from exposure to dust, but the lamp housing was not airtight and was really not made for exterior use. Thus, the moisture inside the quartz sleeve may have been extremely detrimental to the overall effectiveness of the system. The problem with this scenario was that the light was immediately refracted by the moisture and as the moisture dried, residue from dissolved solids in the water droplets dried on the inside of the sleeve which caused further fouling. It is suspected that the major cause of the lower effective doses obtained in the bioassay was due to the moisture buildup inside the quartz sleeve. The other factors stated earlier were minor compared to the major fouling caused by the lack of an airtight lamp housing.

Once the average effective dose was determined from the once-thru-pass tests, the multiple-pass test could begin. The goal of the multiple-pass test was to show that a dose of 140mW-sec/cm^2 could be obtained and that if this dose was applied that the remaining bacteria would be below levels stated for R-1 reuse. Assuming that each pass through the UV system would yield an effective dose of 8mW-sec/cm2, the water was then recycled through the system enough times to obtain the desired dose. A value of 8 mW-sec/cm^2 was used instead of 6.4mW-sec/cm^2 because when the test was performed, the average dose was determined using only one of the collimated beam test results instead of a

compilation of all four tests. Therefore, based on an effective dose of 8 mW-sec/cm^2, 17½ passes would yield an effective dose of 140 mW-sec/cm^2. Figure 26 is a graph of this test. Samples for this test were collected for the first five passes, the tenth pass, and the 17½ pass. This graph shows that the requirement to meet R-1 quality water based on



fecal coliform counts was below 2.2CFU/100mL by the tenth pass, which correlates to a dose of 80mW-sec/cm^2. Also this test was performed on water that had a turbidity of 5.4NTU. Unfortunately, this test did not actually create R-1 quality water since the turbidity was still above 2NTU, but it did show that the disinfection criteria were definitely satisfied.

The results from the multiple-pass test related well to similar tests performed in other experiments. First, the case study outlined earlier showed that fecal coliform counts of less than 2.2CFU/100mL were consistently met with an effective dose of 112mW-sec/cm^2 (Braunstein et al., 1996). Also, a study in 1993 showed that the criteria of less that 2.2CFU/100mL was met with a minimum UV dose of 97mW-sec/cm^2 using total coliform as the tested organism (Darby et al., 1993). Both of these earlier

experiments relate well with the results from this experiment which showed that a dose of 80mW-sec/cm^2 was sufficient to yield fecal coliform counts below 2.2CFU/100mL.

4.4 Results of R-2 Quality Water Test

The attempt to create R-2 quality water using chlorine tablets failed. As explained earlier, the WTPU has a small tank section (approximately 5.8 gallons) at the end of the unit specifically designed for disinfection purposes. This chamber receives all water from a small weir that directs the entire flow path through a perforated container containing 3" diameter chlorine tablets. The 7 oz. tablets are 99% Trichloro-S-Triazetrione and are designed to disintegrate leaving a 1-3 mg/L chlorine residual in pool water. Additionally, each tablet is expected to last for 10,000 gallons when fully submerged in a pool chlorination chamber.

Based on the above data, it is questionable if the chlorine tablets will be able to disinfect the secondary effluent to below 23CFU/100mL. First, the contact time is much shorter than what would be ideal. For example, if the flow of 400gpd was evenly spread throughout the day, this would yield a contact time of 21 minutes in the final contact chamber. However, since the flow is concentrated in three time periods, the contact time is much shorter. The worst case scenario for loading schedule of the test would be the 1700-2000 period which would give a contact time of approximately 6½ minutes. For the testing performed, samples were taken during the morning flow period which had a chlorine contact time of about 7½ minutes. The contact time in this situation is far shorter that traditional values and may not be sufficient in providing doses to meet R-2 quality water standards.

Secondly, the quality of the water in the WTPU is far different than the quality of water in a pool. The manufacturer's data stated that the tablet would give a 1-3 mg/L chlorine residual. This statement assumes that the pool water is potable and either has

almost no chlorine demand or already has a chlorine residual from the water treatment plant. The secondary effluent from the WTPU on the other hand, will most definitely have a chlorine demand, which could even exceed the 1-3 mg/L residual that the chlorine tablet provides. Thus, it is poor logic to assume that a pool tablet would provide the desired disinfection.

Finally, the design of the weir system for the WTPU is not suited for the disintegration rate of the chlorine tablets. Since the chlorine tablets are designed for pool use, they are meant to be placed in a pool chlorination chamber. This chamber forcefully recycles the pool water through the chamber where the chlorine tablet is fully submerged at all times. This is very different from the weir setup in the WTPU in which the water slowly flows around the chlorine tablet, and the tablet is not fully submerged. Therefore, it is highly unlikely that the chlorine tablets will disintegrate at the same rate as in a pool chlorination chamber. In other words, it is unlikely that the tablets will provide a chlorine residual of 1-3 mg/L.

To test the effectiveness of the chlorine tablets, 5 samples were taken at evenly spaced time intervals during the morning loading period. The first sample represented a chlorine contact time of approximately 10 hours since the tank had not been loaded since the previous evening. The last sample represented the shortest chlorine contact time which was approximately 7½ minutes. The three intermediate samples were taken to determine if there was a change in the disinfection effectiveness as the contact time varied throughout the loading period. Additionally, random samples were tested onsite for total chlorine and chlorine residual using a Hach chlorine test kit.

The results of the testing proved that the chlorine tablets did not work in the same way as in a pool situation. There was no noticeable reduction in the fecal coliform count. The results of the microbial testing showed that fecal coliform counts were similar to typical values obtained prior to any disinfection during the UV disinfection period of testing. Additionally, the average total chlorine measured in the disinfected effluent averaged at 0.15mg/L, and there was no measurable free chlorine in the effluent.

The results showed two things. First, the results showed that the chlorine tablets were not disintegrating at the same rate as the manufacturer intended, otherwise, the total chlorine would have been between 1-3mg/L. Secondly, the results showed that there is a chlorine demand, and the chlorine demand is greater than the chlorine dose that is being applied by the chlorine tablets. Clearly, the chlorine tablets do not provide enough of a chlorine dose to disinfect the effluent to R-2 quality water standards.

In order to improve the chlorine disinfection process at the end of the WTPU, two revisions must be made. First, the chlorine contact chambers must be resized to provide a minimum contact time of 15 minutes as stated by the DOH guidelines. Thus, using the loading schedule outlined by NSF Standard 40, and a total daily loading of 400gpd, the disinfection chamber must be at least 14 gallons. This will accommodate the largest loading of 160 gallons in three hours during the evening loading period. Secondly, different chlorine tablets must be used if the existing weir configuration was to remain. The chemical binder used in the tablets would need to release the chlorine into the water at a faster rate. If this was not possible, or using existing chlorine tablets was desired, the weir configuration would need to be revised to submerge multiple chlorine tablets simultaneously in order to raise the total and free chlorine levels in the contact chamber.

Chapter 5
Discussion

Now that the results have been presented, it is necessary to discuss some of the design changes and plans for a real OWRS. Before discussing this, however, a break down of the guidelines that could be changed and those that must be complied with in order to make onsite water reclamation a reality must be understood. First, the regulations that are currently in the guidelines that could possibly be revised or relaxed are discussed. These are the regulations that only apply to current technological limitations or current processes that have driven the regulations in the past. Secondly, some of the guidelines are discussed that, regardless of the improvements made to the system, are unlikely to change. These are the requirements currently listed in the guidelines that state officials feel are necessary to maintain minimum standards that cannot be revised. After discussing how the regulations need to be altered, it will be possible to discuss the actual changes to the system and how the OWRS could possibly be designed. Finally, some of the roadblocks that will remain even if some of the guidelines were revised to accommodate an OWRS are discussed.

5.1 Guideline Revisions

There are some currently active regulations that could be revised to make onsite water reclamation more plausible. The first regulation recommended for revision is the one that requires that signs shall be posted where reclaimed water is used. As stated earlier, this regulation has an extremely negative psychological impact on people unfamiliar with water quality standards. A sign stating that reclaimed water is used for

irrigation around the home could be a major deterrent for this type of system. Images of horrible diseases or even death come to mind for the uninformed person and could be a real source of headache and trauma for a homeowner relating to friends and neighbors. On the other hand, the alternative of keeping the homeowner and neighbors uninformed would also be a poor choice. For example, current water supply systems allow drinking from an outdoor hose or sprinkler head, however, if an onsite water reclamation system was used this practice would not be recommended. Thus, there would be some level of education necessary to the general public before R-1 quality water could be used regularly in a residential area. In other words, a sign in everyone's front yard may not be the solution; but neither would be the lack of notification. The challenge of getting the general public to accept this system may possibly be the largest drawback to making water reuse possible at a residence.

The regulation concerning the maintenance and care of a system is the next guideline requiring revision prior to onsite water reclamation. The DOH guidelines currently state that reclaimed water may only be used at a residential property when managed by an irrigation supervisor. It seems as if the intent of this guideline is for the management of reclaimed water at large, privately owned properties, such as golf courses, or farms. This guideline is not practical, however, from a single-homeowner's point of view. Yet, while it is unreasonable to expect a homeowner to hire an irrigation supervisor, it may also be unreasonable to expect a homeowner to be in charge of the maintenance of the system because of the technical nature of the system and the lack of any assurance to the regulators that the system would be properly maintained. Ultimately, this determination would depend upon the level of monitoring required by the

system. One recommendation to revise this rule would be to designate the company selling the wastewater treatment package unit and the onsite water reclamation system as the irrigation supervisor. This may prove to be acceptable since the WTPU would require regular maintenance, and thus, a reclaimed water use program could be incorporated into the regular maintenance program for the WTPU. This relieves the owner of having to ensure that water meeting R-1 standards is produced. Also, a regular maintenance program would be much less expensive for the homeowner than hiring a full time individual to maintain this system. The only drawback to this recommendation, however, is that onsite water reclamation is still very costly when compared to the cost of potable water today. Currently in the state of Hawai‘i, potable water costs approximately $1.17/1,000gallons. Therefore, just to make the maintenance program comparable to the current cost of potable water, the monthly maintenance bill would need to be under $20-$30/month, the cost of a typical water bill.

The third regulation that would require revision in order to make onsite water reclamation more plausible is the requirement that a UV disinfection system must have three UV banks in series to provide a minimum design dose of 140mW-sec/cm^2 at a maximum week flow. The current rationale for requiring three UV banks in series is that three banks is considered to be the minimum number of banks necessary to prevent short circuiting and inadequate disinfection (White, 1999). This rationale is designed to ensure that if a lamp failure occurs or other problems prevent one bank from operating at a less than ideal condition, an adequate dose could still be provided to the microorganisms. This is definitely a necessary criterion for a wastewater treatment plant which services a large amount of water and must discharge it before additional water can be accepted at

the plant, but this is not the case for a single home OWRS such as this one. For example, if the onsite water reclamation system was designed with only one lamp and there was a lamp failure, the water could be diverted to the leach field and the WTPU could operate in a similar fashion as a septic tank. Thus, the requirement of having back up UV banks is really a luxury for this system and not at a necessity at all. In essence, all that should be required in the guidelines is that the microbial requirements for R-1 water be met before use. Furthermore, the methodology on how this is accomplished should be at the manufacturer's discretion.

A regulation associated with the requirement of three UV banks in series is the requirement for backup power to be supplied to the system. If it is understood that in the event of a lamp failure that R-1 water production would temporarily be suspended, then the same reasoning could be employed for power failures. Therefore, it is recommended that a backup power supply system for the UV disinfection process be eliminated.

Another unnecessary regulation is the additional requirement for minimum turbidity levels prior to filtration. The current regulation requires that the secondary effluent used for reclamation must be below 5NTU before filtration. The rationale behind this regulation is that conventional wisdom states that secondary effluent above 5NTU cannot reasonably be treated to meet the final requirement of below 2NTU. Therefore, the State DOH placed this requirement in the guidelines as a goal for secondary effluent quality. Depending on the type of sand filter used and the use of chemical coagulants, however, the final turbidity requirement can be achieved with secondary effluents greater than 5NTU. Thus, in order for this system to operate consistently, the initial turbidity requirement for secondary effluent must be relaxed.

Again, this is an intermediate regulation that does not alter the final water quality, and is therefore unnecessary.

Finally, the criteria for alarms must be reduced if an OWRS is to be possible. The guidelines for UV disinfection require eight different alarms throughout the process. Alarms are required to monitor: flow rate, transmittance, turbidity, liquid level, on/off status of each UV bank, on/off status of each UV lamp, UV intensity, and lamp age. As in the case of some of the guidelines already discussed, some of these requirements do not apply. Of these eight alarms, only three apply to this design. First, there would be little reason to monitor the flow rate since this system is not dependent on an adjustable baffle system like most UV systems which regulate flow and the liquid level in the flow channel. Rather, this system would operate on a pump that has only one flow rate and pumps the water through and enclosed UV system. Therefore, a flow meter alarm would be unnecessary, as would an alarm to monitor the liquid level of each channel. The on/off status of each UV bank would also be unnecessary since it was recommended earlier that only one UV lamp would be used. Thus, the alarm to monitor the status of the UV lamp would be sufficient in this system. Finally, the lamp age and UV intensity should not be required in this system since the system would be designed to the minimum dose produced over the life of the lamp. Therefore, an alarm would only be needed to monitor the on/off status of the lamp, signaling when R-1 quality water was not being produced. This leaves alarms for the transmittance, turbidity, and status of the UV lamp as the only ones necessary for the operation of this system.

It is possible that even the transmittance would not require an alarm since this study shows that every time the turbidity was too high, the transmittance was too low;

and never was the transmittance above the guidelines when the turbidity was within acceptable limits. Based on previous results, however, this might not always be the case. Two separate studies showed that there was a poor correlation between transmittance and turbidity (Darby, 1993; Petrasek et al., 1980). In contrast, one showed an excellent correlation between the two (Harris et al., 1987). A possible reason for the confusion in the relationship between these two measurements is the fact that transmittance is effected more by the dissolved constituent in the water than the suspended particles (Qualls et al., 1983). Thus, until the relationship between transmittance and turbidity is better understood, both will require monitoring.

5.2 Guidelines Remaining

So far it seems as if there would need to be many revisions to the existing guidelines in order to make onsite reclamation possible, but there are also many regulations that could remain unchanged. Regulators would assume that a lower level of safety would occur if revisions to the following guidelines were made. The first guideline that would unlikely change is found in the statement made in the DOH guidelines that adequate measures should be made to prevent the ponding of reclaimed water. This regulation is very similar to the next one in the guidelines which states that reclaimed water should be managed to avoid the proliferation of mosquitoes. Both of these regulations are concerned with the possibility of any pathogenic bacteria growth that may remain in the area. The concern would be that undesirable microorganisms could proliferate in ponded water, infect mosquitoes, and spread disease. Since this regulation would not effect the design of the system, however, revisions to the guidelines and the OWRS would not be required.

These two regulations, however, shed light on how the use of reclaimed water is viewed. Clearly, regulators have serious apprehensions about the use of reclaimed water, making it difficult to implement and enforce the proper use of reclaimed water in a residential zone. This apprehension is heightened because homeowners do not carefully monitor their water usage the way a golf course, farm, or larger organization would. This is not to say that homeowners are careless, but simply that the construct of society does not currently require homeowners to monitor their water use. There are no rules for watering a homeowner's lawn or carefully preventing ponding in one's yard. Therefore, suddenly requiring these things to be taken seriously may be beyond the desire of many

homeowners. The inconvenience caused by using reclaimed water may alone be enough to deter many homeowners from using such a system even if they could save money.

The next regulation unlikely to change is the requirement that no discharge, runoff, or overspray may spread beyond approved boundaries. The rationale for this guideline is that it is impossible for regulators to know what conditions exist beyond the boundaries of the person using the reclaimed water. This guideline may just be an artificial restraint, however, since some sort of boundary limitations had to be described in the guidelines. Thus, the practicality of the OWRS could heavily depend on how this guideline is followed. If this guideline was strictly enforced, sprinkler irrigation would be nearly impossible since sprinklers always cause overspray and runoff to some extent. Realistically, however, it is unlikely that the guideline would be interpreted this way. If it was determined that reclaimed water was safe for one homeowner at one residential property, then it would be unlikely that overspray onto an adjoining property would create unsafe conditions for the neighboring homeowner. Therefore, this guideline should most likely not cause any additional difficulty in making onsite water reclamation a reality.

Also, there are two regulations concerning the use of reclaimed water near drinking water supply wells. The first requirement states that there can be no irrigation using reclaimed water within a minimum of fifty feet from a drinking water supply well. The second requirement states that the outer edge of the impoundment should be one hundred feet from any drinking water supply well. These requirements do not directly relate to the reclaimed water concepts discussed in the majority of this paper since they do not necessarily apply to water reclamation near most residential units because most

homes do not have their own drinking wells. Therefore, these two requirements apply more to a rural setting such as a farm or a home in a remote location and do not directly apply to onsite water reclamation in a residential zone.

Finally, it is highly unlikely that any of the biological criteria or water quality standards would be relaxed to accommodate an OWRS. The responsibility to meet standards such as a final effluent quality of 2NTU and less than 2.2CFU/100mL for R-1 water, or less than 23CFU/100mL for R-2 water is strictly on the shoulders of the system designers. Thus, any regulations pertaining to these standards should be left out of any conversation with the State DOH when presenting this system for review. Additionally, since the regulations are not structured to accommodate the type of system described in this report, it is safe to assume that there may be some additional requirements that could arise to regulate an OWRS. Therefore, restrictions currently not in the guidelines should be expected to regulate an OWRS.

5.3 Proposed Design

Now that the regulations have been discussed and the testing results have been presented, it is questionable if an OWRS to produce R-1 quality water is currently practical. First, the WTPU / OWRS package is currently not reliable enough to consistently produce R-1 quality water. Secondly, many revisions to the existing guidelines are necessary to make the system a reality. Finally, if the guidelines were strictly enforced, water overspray and runoff would be almost impossible to avoid. While R-1 quality water is difficult to currently produce, R-2 quality water is not. The system (as currently constructed) was shown to easily produce water that met the R-2 quality standards (see Figure 26). Also, if the guidelines were strictly enforced and drip irrigation was the only practical form of irrigation, R-2 water could still be used since this is the approved form of irrigation for R-2 water in a residential zone. In summation, an OWRS to create R-2 quality water is ideal since it requires less revisions to the existing guidelines and is more in line with the current capabilities of the OWRS.

This section will first discuss the recommended design for an actual OWRS. Secondly, a maintenance program to care for the system and ensure that the guidelines are consistently met will be discussed. Finally, the last section will address some of the remaining problems with the system even after the recommended revisions have been incorporated.

When designing a follow-on system to this experiment, it is highly recommended that an upflow sand filter be used. This is recommended for many reasons. First, the case study (Braunstein et al., 1996) showed that an upflow sand filter very effectively removed suspended solids to levels below 2NTU. This pre-made sand filter effectively

allowed the researchers to focus on the UV disinfection and not have to bother with many of the problems encountered in this project. Secondly, an upflow sand filter does not require periodic backwashing since it is continuously backwashed. This reduces the regular maintenance required on the sand filter and does not create a surge of backwash water into the WTPU. Also, the upflow filter is better suited for the configuration of this system. As shown in Figure 3, this experiment relied on the existing elevation of the secondary effluent from the WTPU. The water flowed via gravity through the sand filter, and the UV system into the collection container. Although this is an easy and inexpensive way to run the system, this method would be full of problems. The main flaw with a gravity design is that the final WTPU would be installed underground. This would mean that the filter, UV lamp, and collection container would also have to be underground if a gravity rapid sand filter was used. This would be extremely impractical. Since the system would be underground, maintenance on the sand filter would be nearly impossible, an effective backwash of the filter could not be performed since visual inspection is practically required, and the UV lamp would be completely inaccessible for maintenance. Thus, we are left with a system that requires a pump to bring the secondary effluent to the surface. Therefore, since a pump is already required in the design, an upflow filter might as well be used instead of a gravity filter.

The second major revision required to improve this system is to have an airtight UV system. This revision is needed because the accumulation of moisture on the inside of the quartz sleeve was the most likely cause of fouling in the UV system. The case study mentioned earlier stated that there was little change in the effectiveness of the UV lamps when the quartz sleeves were cleaned daily versus when they were not cleaned for

ninety days. Also, since our testing period lasted only slightly longer than the average recommended cleaning period (one month) for a quartz sleeve, it is highly unlikely that the outside of the sleeve was extremely dirty. This fact was also noted after the testing period when the quartz sleeve was removed and inspected for contamination. Little dirt, water marks or bacterial growth were observed on the outside of the quartz sleeve. Thus, the major cause of fouling was likely due to the moisture build up on the inside of the quartz sleeve because the UV system was not airtight.

The revisions necessary to make an airtight system are quite simple. Currently, the quartz sleeve is placed into the reactor, followed by a threaded aluminum cylinder, which secures the quartz sleeve in place and guides the UV lamp into location when installed. After the UV lamp is installed, a rubber gasket is fastened over the aluminum cylinder. This gasket provides excellent protection from water and dust that could enter and effect the performance of the lamp, but this current configuration is not airtight. To make the system airtight, the UV lamp and quartz sleeve must be manufactured as a single unit. Then there would be no chance of moisture accumulating on the inside of the quartz sleeve. This would require only small revisions to the current design of the UV system and would greatly reduce the chances of fouling due to moisture. Also, the quartz sleeve, which is sealed around the lamp, would provide greater protection to the lamp. The main disadvantage of this design would be the additional cost associated with manufacturing the lamp/sleeve package versus manufacturing them as separate items. This design would also effect the cost of repair when lamp replacement is required. Due to the necessity of an airtight system, however, this additional cost cannot be avoided.

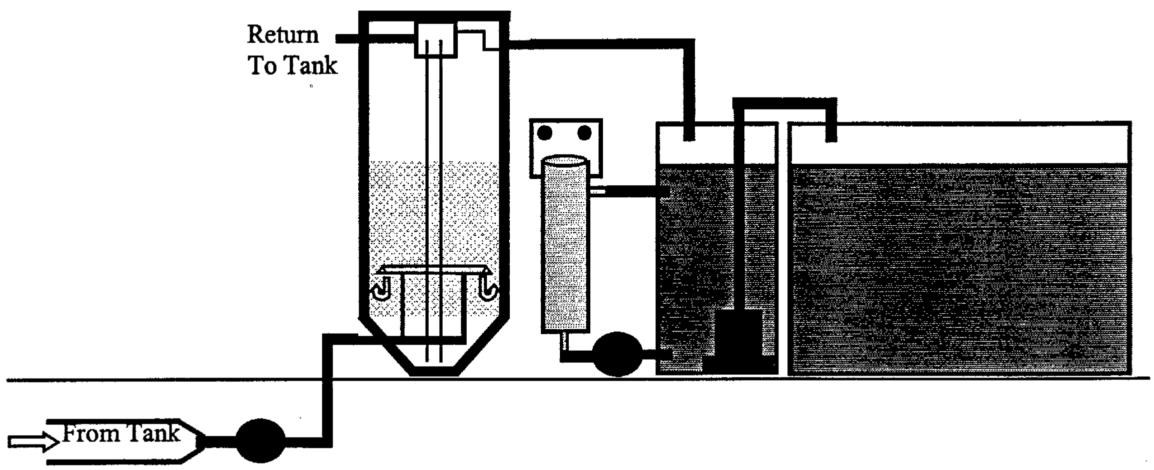
Another necessary revision to this system would be the overall process by which the reclaimed water is made. The system that was constructed in this experiment collected the filtered water in a single collection container. To achieve the regulatory dose of 140mW-sec/cm^2, the water pump pushed water through the UV reactor and discharged the water back into the same collection container, as explained in an earlier section. This recycling process could effectively treat a set amount of filtered effluent and provide the necessary effective dose to meet current DOH guidelines. The problem with this design, however, is that the system does not contain a final holding tank for the R-2 quality water. A separate tank would definitely be required since the sand filter directly discharges newly received effluent from the WTPU into the current collection container. Thus, as the system is currently built, whenever new filtered effluent is discharged into the collection container, the entire amount of the effluent in the container must be recycled through the UV reactor. Also, if this was the only collection container in the OWRS, whenever the recycle process was operating, no water would be available for reuse.

The following design is proposed to remedy the current problems. First, a twenty to fifty gallon container would be required to collect filtered effluent from the WTPU. Once the container is full, an electronic valve would be required to divert all secondary effluent from the WTPU into the leach field until the ultraviolet disinfection process was complete. Then, all disinfected effluent would be transferred to a final collection container which could then be used for reclaimed water purposes. Finally, the electronic valve would be reopened to redirect all secondary effluent back into the sand filter to repeat the process. This batch process for creating R-2 quality water loses any secondary

effluent that is discharged during the recycle process, but the losses should be minor compared with the amount collected. This specific problem could also be avoided if two intermediate containers were used, but this option is not ideal due to the added overall cost and size of the system.

Figure 27 is a schematic of the OWRS with the recommendations explained

Figure: 27 Proposed Design



already incorporated into the design. This figure shows that the system would be above ground while the WTPU would be below ground. Therefore, a pump would be required to lift all secondary effluent into the sand filter. An upflow sand filter is incorporated into the design since a pump is already required and since an upflow filter would require less maintenance and servicing than other filter types. Following the filter, the intermediate collection container receives all water from the sand filter until its capacity is reached. Once this capacity is reached, the container and UV system begin the recycle process until the required effective dose is obtained. Then a submersible pump transfers all of the disinfected water from the intermediate collection container to the final

collection container. Finally, the water is ready for reuse; and the filtration/disinfection system repeats the entire process with a new batch of secondary effluent.

5.4 Remaining Problems

Even if all of the above changes were made to the system; and the DOH guidelines were revised to accommodate the OWRS, some problems would still remain. The first problem is that the cost of the system outweighs any of the cost savings that could be realized by this system. This is true because the only cost savings that can be realized by using an OWRS is the savings in monthly potable water use. At best, potable water use reduction for a four-person family would be 60% of the total water used (see Figure 2). In Hawaiʻi, the average four person family's water bill is approximately \$20-\$30/month. Therefore, the possible savings that could be realized using an OWRS would be \$12-\$18/month. Compared with the monthly costs that would be required to operate the OWRS, the potable water savings would be minimal. Not only would the system have a high initial cost, but the monthly operating costs would also be high. As stated earlier, the quartz lamp sleeve for the UV disinfection unit would need a monthly cleaning. Even if this was the shortest time period that maintenance calls were made, one maintenance call per month would cost at least \$60. This already exceeds the monthly savings in potable water and doesn't even include repairs to the system, new parts, electricity cost or any regulatory testing. Clearly, the cost would still be a major issue.

The second problem remaining with this system is the maintenance schedule itself. The estimate of \$60/month is a best case scenario. In reality, however, this cost could be much more. A maintenance program would need to periodically check the condition of the valves, filter, UV system, pumps, UV lamp, and UV sleeve. Ideally, these systems should be checked at least once a week to make sure that the system is operating within the required parameters; but this would greatly increase the cost of a

maintenance program. Another question that has been left unanswered until now is the question of the number of microbial tests that would be required to ensure R-2 quality water. The guidelines state that the disinfected effluent must not exceed 23 CFU/100mL utilizing bacteriological results of the last seven days and that the density does not exceed 200 CFU/100mL in more than one sample in any 30 day period. The guidelines are written in a way such that the intent seems to require a daily microbial test. Daily testing, however, certainly excludes any maintenance program from being even slightly realistic in cost. This requirement is similar to the requirement that restricts overspray and discharge beyond approved boundaries in that it will likely not be strictly enforced. Thus, daily testing would probably not be the case and this would be especially true for R-2 quality water and a drip irrigation system since the risk of human contact with the water is low. Also, since the system can easily disinfect the water to meet the DOH guidelines, the likelihood of less frequent testing is also increased.

The last problem with this system is that it would not be as safe as the residential practices that are currently done now. Even if the OWRS was working perfectly and was creating R-2 quality water consistently, the water created is not as clean as current drinking water. Therefore, the water is not as safe as potable drinking water. This is something that communities and regulators may have difficulty accepting. Once a certain level of safety has been set, it is difficult to lower the standard. In addition, the current level of safety requires little effort or knowledge on the part of the homeowner. If suddenly, the level of safety were to decrease and the level of knowledge or training to operate an OWRS were to increase, the desire to use reclaimed water might be less. Thus, onsite water reclamation is ultimately controlled by the desires of the homeowner.

Since onsite water reclamation is ultimately controlled by the desires of homeowners, however, desires can be changed as the conditions change. For example, if the cost of potable water increases or water shortages restrict irrigation, water reuse will look more attractive. Eventually the desire to save money or have a green lawn may outweigh any fears of reclaimed water being unsafe. Also, government subsidies to offset the additional cost of an OWRS or educational programs to inform the general public of the benefits of water reuse may change the public's opinion about water reuse in a residential area. In conclusion, current public opinion and current cost restrictions are only current barriers. Inevitably the current conditions will change and thus, these barriers will change making onsite water reclamation much more possible, attractive, and cost effective in the future.

Chapter 6
Conclusion

The following recommendations are made to the OWRS design:

- The OWRS should be designed with the purpose to create R-2 quality water.
- The sand filter should be a continuously backwashed, deep bed upflow sand filter.
- An airtight UV system is crucial to efficient system performance.
- A batch system process for creating R-2 quality water is recommended.

The following recommendation is made concerning revisions to the State DOH guidelines for water reuse:

- Intermediate requirements should be removed. DOH guidelines should only outline the final requirements necessary for the classification of the desire type of reclaimed water for small onsite systems.

Appendix

Date	Total Suspended Solids (mg/L)			Volatile Suspended Solids (mg/L)		
	Before Filter	After Filter	UV (Once)	Before Filter	After Filter	UV (Once)
3/4/99	2.67	5.25		2.67	3.23	
3/5/99						
3/9/99	10.00	8.80	7.10	8.60	6.20	4.90
3/18/99	7.50	4.65	3.26	6.46	2.83	1.63
3/19/99	4.12	1.47	3.33	2.87	0.63	1.46
3/22/99	3.40	1.00	1.82	3.00	1.00	1.00
3/24/99	5.35	1.56	1.59	4.19	1.25	1.14
3/25/99	5.40	1.88	1.80	4.80	1.56	1.35
3/26/99	6.53	2.57	3.20	5.71	1.71	2.20
4/2/99.	3.31	1.67	2.04	2.90	1.04	1.22
Averages	4.80	1.74	2.09	4.12	1.31	1.38

Date	Turbidity (NTU)				Transmittance (%)			
	Before Filter	After Filter	UV (Once)	UV (Twice)	Before Filter	After Filter	UV (Once)	UV (Twice)
3/4/99	5.8	4.8			59.9	60.5		
3/5/99	5.5	4.5			61.6	62.4		
3/9/99	10	10	10		43.3	39.9		
3/18/99	7.2	5.2	5.9		53.5	57.8	58.4	
3/19/99	6.9	5.1	5.8		52.1	56.1	57.7	
3/22/99	5.9	2.6	2	2.4	65	72.1	72	71.4
3/24/99	7	3.3	2.7		53	59.5	62.6	
3/25/99	8	4.6	4		48.3	54.1	56.5	
3/26/99	8.8	5.7	5.2		45.3	48.3	50.3	
4/2/99	7.8	5.4	5.5		51	55.7	56.3	57.2
Averages:	7.5	4.3	3.9		52.5	57.9	59.5	

Date	Density (CFU/100ml)				Test Comments
	Before Filter	After Filter	UV (Once)	UV (Twice)	
3/4/99	1,700,000	1,240,000			System backwashed 2 days ago
3/5/99	1,500,000	1,520,000			
3/9/99	8,000,000	7,200,000	478,000		
3/18/99	600,000	85,000	1,500		Added 4" 90 grit sand to filter
3/19/99	200,000	250,000	7,500		
3/22/99	290,000	60,000	600	50	Sand filter backflushed and now 12" in depth
3/24/99	2,120,000	1,700,000	17,000		
3/25/99	1,440,000	1,162,000	9,800		
3/26/99	1,740,000	1,260,000	23,800		
4/2/99	1,280,000	1,000,000	3,000	1,160	Backwashed Filter before the test

Collimated Beam Test #1		
Date:	2/10/99	
Dose (mW-sec/cm^2)	Density (CFU/100ml)	Log(N/No)
0	2,226,667	0.0000000
5	284,000	0.8943369
10	5,213	2.6305397
40	6	5.5695040
50	15	5.1813238
75	6	5.5695040

Collimated Beam Test #2		
Date:	3/24/99	
Dose (mW-sec/cm^2)	Density (CFU/100ml)	Log(N/No)
0	1,700,000	0.0000000
5	73,000	1.3671261
10	503	3.5288809
20	37	4.6622472
30	7	5.3853509
40	3	5.7533277
50	8	5.3273589
75	9	5.2762064

Collimated Beam Test #3		
Date:	3/25/99	
Dose (mW-sec/cm^2)	Density (CFU/100ml)	Log(N/No)
0	1,162,000	0.0000000
5	100,800	1.0617456
10	540	3.3328124
20	15	4.8891149
30	6	5.2870549
40	2	5.7641761
50	5	5.3662361
75	2	5.7641761

Collimated Beam Test #4		
Date:	3/26/99	
Dose (mW-sec/cm^2)	Density (CFU/100ml)	Log(N/No)
0	1,260,000	0.0000000
5	51,200	1.3911006
10	248	3.7059189
20	5	5.4014005
30	8	5.1972806
40	9	5.1461280
50	5	5.4014005
75	4	5.4983106

Multiple Pass Test (4/2/99)				
Sample Description	Received Dose (mW-sec/cm^2)	Density CFU/100mL	Pass #	Log (N/No)
Original	0	1,000,000	0	0
Pass 1	8	3,000	1	2.52
Pass 2	16	1,160	2	2.94
Pass 3	24	307	3	3.51
Pass 4	32	105	4	3.98
Pass 5	40	12	5	4.92
Pass 10	80	1	10	6
Pass 17.5	140	1	17.5	6

Alum Test		
Date: 4/16/99	Dose ~ 75mg/L	
Sample	Turbidity	% Trans.
Original effluent	5.0	
After alum	4.0	62.7
After filter	2.4	72.5
After UV (once)	3.3	72.5

Alum Test		
Date: 4/20/99	Dose ~ 75mg/L	
Sample	Turbidity	% Trans.
Original effluent	5.9	55.3
After alum	5.3	60.0
After filter	3.3	66.0
After UV (once)	3.7	65.1

Alum Test		
Date: 4/23/99	Dose ~ 200mg/L	
Sample	Turbidity	% Trans.
Original effluent	10.5	43.8
After alum	1.7	73.2
After filter	3.4	70.0
After UV (once)	3.9	69.0

Jar Testing			
Alum Dose (mg/L)	Turbidity	% Removal	
0	7	0	Date: 4/21/99
15	5.3	24.3	
30	3.6	48.6	
45	3.3	52.9	
60	2.2	68.6	
90	1.9	72.9	
120	1.1	84.3	
150	0.73	89.6	Date: 4/22/99
180	0.67	90.4	
210	0.5	92.9	***Choose optimum dose of 200 mg/L**
240	0.46	93.4	
270	0.41	94.1	
300	0.43	93.9	

Particle Size Testing		
Date: 4/20/99		
Sample	Turbidity	% Trans.
<u>Original effluent</u>	5.9	55.3
5μm filter	2.6	61.4
1.2μm filter	1.8	65.1
0.8μm filter	2.1	63.8
Glass fiber filter	4.1	56.9
<u>After alum</u>	5.3	60.0
5μm filter	1.7	67.6
1.2μm filter	1.0	71.9
0.8μm filter	1.3	70.7
Glass fiber filter	2.6	64.4

Chlorine Testing		
Date: 5/19/99		
Parameter	Value	
Turbidity	30 NTU	
Total Chlorine	0.2mg/L	*Hach test
Free Chlorine	0.0mg/L	*Hach test
Fecal Coliform Count	2,160,000 CFU/100ml	

Chlorine Testing		
Date: 5/27/99		
Parameter	Value	
Turbidity	64 NTU	
Total Chlorine	0.1mg/L	*Hach test
Free Chlorine	0.0mg/L	*Hach test
Fecal Coliform Count	8,160,000 CFU/100ml	

Turbidity(NTU) unfiltered			
Test #	Day	Influent	Effluent
1	10/23/98	32	1.5
2	10/26/98	33	1.6
3	10/27/98	42.5	1.4
4	11/6/98	44	3.3
5	11/9/98	36	3.5
6	11/10/98	43	3.5
7	11/11/98	44	3.7
8	11/12/98	45	4.2
9	11/13/98	63	5.0
10	11/17/98	54	4.4
11	11/18/98	55	4.0
12	11/26/98	43	4.1
13	11/27/98	32	8.8
14	11/30/98	46	6.6
15	12/8/98	37	6.2
16	12/25/98	60	6.5
17	12/10/98	22	5.1
18	12/25/98	60	6.2
19	12/28/98	53	7.0
20	12/29/98	74	5.4
21	12/30/98	74	6.5
22	12/31/98	52	5.2
23	1/1/99	22	5.1
24	1/4/99	28	4.3
25	1/5/99	42	3.9
26	1/6/99	35	3.5
27	1/7/99	30	3.0
28	1/8/99	35	3.5
30	1/12/99	55	3.5
31	1/14/99	52	4.2
32	1/15/99	67	3.9
33	1/18/99	45	5.6
34	1/19/99	48	6.1
35	1/21/99	58	7.3
36	1/22/99	57	6.5
37	1/25/99	47	4.2
38	1/26/99	65	4.0
39	1/27/99	34	3.3
40	1/29/99	68	3.5
41	3/1/99	84	8.2
42	3/2/99	65	7.4
43	3/3/99	72	6.6
44	3/4/99	70	5.8
45	3/5/99	65	5.5
46	3/9/99	70	10.0
47	3/8/99	71	10.0
48	3/10/99	51	7.5
49	3/12/99	60	30.0
50	3/15/99	65	5.7
51	3/16/99	52	2.6
52	3/17/99	55	4.0
53	3/18/99	65	7.2
54	3/19/99	60	8.2
55	3/22/99	50	4.0
56	3/23/99	62	3.7

Turbidity(NTU) filtered			
Test #	Day	Influent	Effluent
1	11/20/98	8.4	1.2
2	11/23/98	8.5	1.3
3	11/25/98	23	1.9
4	12/1/98	16	3.7
5	12/2/98	38	2.7
6	12/3/98	36	3.6
7	12/8/98	15	4.5
8	12/11/98	15	4.1
9	12/14/98	7.7	3.4
10	1/4/99	25	2.8
11	1/5/99	22	2.0
12	1/6/99	24	2.0
13	1/7/99	22	2.0
14	1/8/99	25	2.3
15	1/11/99	25	2.1
16	1/12/99	32	2.0
17	1/13/99	29	2.1
18	1/14/99	18	2.1
19	1/15/99	40	2.2
20	1/18/99	17	3.1
21	1/19/99	4.7	4.0
22	1/21/99	24	5.0
23	1/22/99	34	4.2
24	1/25/99	32	2.4
25	1/26/99	23	2.1
26	1/27/99	19	2.0
27	1/29/99	35	2.0

Transmittance (filtered)			
Time#	Day	Influent	Effluent
1	11/9/98		60.5
2	11/10/98		60.1
3	11/11/98		60.8
4	11/12/98		56.8
5	11/13/98		63
6	11/16/98		62.6
7	11/17/98		62.3
8	11/18/98	40.1	63.3
9	11/19/98	39.6	65.1
10	11/23/98	33.6	67.4
11	11/24/98		65.6
12	11/25/98	46.5	62.2
13	11/26/98		59.2
14	11/27/98		42.2
15	12/1/98	37.1	52.6
16	12/2/98	30.9	55.9
17	12/3/98	35.2	54.6
18	12/7/98	38.5	52.1
19	12/8/98	38.4	52.7
20	12/10/98	38.1	51.9
21	12/11/98	37.2	53.3
22	12/14/98	35.1	54.9
23	12/15/98	50.0	56
24	12/16/98	52.1	62
25	12/17/98	43.5	66.8
26	12/18/98	40.8	65.5
27	12/21/98	44.0	56.9
28	12/22/98	49.6	60.4
29	12/23/98	42.3	65.1
30	12/24/98	36.1	54.2
31	12/25/98	44.6	47.2
32	12/28/98	36.2	48.5
33	12/29/98	36.0	48.6
34	12/30/98	43.6	46.8
35	12/31/98	30.7	52.1
36	1/1/99	27.5	53.3
37	1/4/99	36.2	57.2
38	1/5/99	35.1	61.5
39	1/6/99	38.9	60.7
40	1/7/99	37.4	61.2
41	1/8/99	34.1	59.4
42	1/11/99	35.4	57.9
43	1/12/99	40.3	60.5
44	1/13/99	39.1	62.7
45	1/14/99	46.9	65.8
46	1/15/99	37.1	64
47	1/18/99	38.1	61.5
48	1/19/99	54.3	60.4
49	1/21/99	48.4	55.7
50	1/22/99	38.0	59.1
51	1/25/99	33.6	65.5
52	1/26/99	44.4	64.6
53	1/27/99	46.6	67.4
54	1/29/99	44.5	68.6

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